Hochschule für Angewandte Wissenschaften Hamburg
Hamburg University of Applied Sciences

Bachelor Thesis

Quynh Chau Nguyen

Risk Based Methodology for Planning Periodic Inspections of Offshore Substations

in accordance with the standard for construction of the German Federal Maritime and Hydrographic Agency

Fakultät Technik und Informatik
Department Maschinenbau und Produktion

Faculty of Engineering and Computer Science
Department of Mechanical Engineering and Production Management

Quynh Chau Nguyen

Risk Based Methodology for Planning Periodic Inspections of Offshore Substations

in accordance with the standard for construction of the German Federal Maritime and Hydrographic Agency

Bachelorarbeit eingereicht im Rahmen der Bachelorprüfung

im Studiengang Maschinenbau/Energie- und Anlagensysteme
am Department Maschinenbau und Produktion
der Fakultät Technik und Informatik
der Hochschule für Angewandte Wissenschaften Hamburg

in Zusammenarbeit mit:
DNV GL – Germanischer Lloyd Industrial Services GmbH
Abteilung Plants & Pipelines – Oil & Gas
Brooktorkai 18
20457 Hamburg

Erstprüfer: Prof. Dipl.-Ing. Peter Dalhoff
Zweitprüfer: Dipl. Ing. Jochen Künzel

Abgabedatum: 08.12.2015

Zusammenfassung

Quynh Chau Nguyen

Thema der Bacherlorthesis

Risikobasierte Methodik für die Planung der wiederkehrenden Prüfungen von Offshore Bauwerken gemäß dem Standard für Konstruktion des deutschen Bundesamtes für Seeschifffahrt und Hydrographie

Stichworte

Wiederkehrende Prüfungen, BSH-Standard Konstruktion, ISO 19902, MSC.1/Circ.1432, risiko-basierte Methode, Ermüdungsanalyse, quasi-statische Analyse, hydrodynamische Belastung, Auslastung der Rohranschlüssen, FEM, Spannungskonzentrationsfaktoren, Wasserlöschsystem, HAZID, Sichtprüfung, Funktionsprüfung

Kurzzusammenfassung

Diese Arbeit umfasst die risikobasierten Methode für die Planung von wiederkehrenden Prüfungen einer Offshore-Umspannstation. Die generellen Grundsätze sind vorgestellt und für das strukturelle System als auch für das Anlagensystem konzeptionell umgesetzt. Außerdem sind die Analysen und Risikobeurteilungen für ein Muster-Jacket und eine typische Wasserlöschanlage durchgeführt. Die Ergebnisse zeigen, dass die risikobasierte Methode eine transparente und gründliche Lösung ist, um die Anlage in ihrem Betriebszustand zu bewerten und um wiederkehrenden Prüfungen zu planen.

Quynh Chau Nguyen

Title of the paper

Risk Based Methodology for Planning Periodic Inspections of Offshore Substations in accordance with the standard for construction of the German Federal Maritime and Hydrographic Agency

Keywords

Periodic inspections, BSH-Standard Construction, ISO 19902, MSC.1/Circ.1432, risk-based methods, fatigue analysis, quasi-static analysis, hydrodynamic load, utilization of tubular members and joints, FEM, stress concentration factors, water extinguishing system, HAZID, visual inspection, function inspection

Abstract

This thesis comprises the risk-based methods for planning periodic inspections of an offshore power substation. The general principles are presented and conceptually implemented for both typical kinds of facilities: a structural system and a plant system. Furthermore, analyses and risk assessments in detail are also conducted for a sample offshore jacket and a sample water extinguishing system. The results point out that the risk-based methods are the transparent and reasonable solution for evaluation systems in working conditions and hence for planning periodic inspections.

Task assignment

Thema

The offshore wind energy is considered as the mainstay of the energy transmission in Germany, the reconstruction of the power systems to renewable energies.

According to standards of the German Federal Maritime and Hydrographic Agency (BSH), periodic inspections are required in order to maintain the project certificate and operating permit.

Due to the complexity of an offshore platform and difficulties to be confronted with along the service life, risk-based methods are recommended for the concepts of periodic inspections.

The topic of the bachelor thesis is: *"Risk-based methodology for planning periodic inspections of offshore power substations in accordance with the standard for construction of the German Federal Maritime and Hydrographic Agency"*.

In this thesis, the basis for calculations in structural analyses as well as methods of quality management for plant systems are presented and applied in examples of an offshore jacket model and a typical water extinguishing system.

Emphases:

- Simplified risk-based methods for planning periodic inspection
- Fatigue analysis
- Plant engineering

Foreword

The current paper is prepared within the framework of thesis for the Degree of Bachelor of Engineering with the emphasis Energy and Plant Systems at the University of Applied Science Hamburg. It is about strategies for planning periodic inspections for an offshore converter platform according to the requirements of The Federal Maritime and Hydrographic Agency (Bundestamt für Seeschifffahrt und Hydrographie - BSH).

This bachelor thesis has been written at the division of oil and gas in the company DNV GL Hamburg during Autumn 2015. It was only possible with the help, time and effort of many colleagues in the departments for offshore installations and for plants and pipelines. Hereby, I would like to express my appreciation to my line manager for giving me the opportunity to conduct the work and to all the people who supported my research and helped me with the creation of this thesis.

First of all, I thank my examiner, Professor Dipl.-Eng. Peter Dalhoff, who gave me substantial feedback and counsels with general as well as technical issues of writing a bachelor thesis in mechanical engineering. I also thank my supervisor, Dipl.-Eng. Jochen Künzel, for his consultancy and guidance during the thesis.

Furthermore, I would like to express my gratitude to my advisors, Dipl.-Eng. Thomas Jahnke and Dipl.-Eng. Hiltraud Möhlmann, whose expertise, understanding and patience added considerably to my graduate experience and technical knowledge.

The very special thanks goes to my friends who provided me with their helpful opinions and editing assistance. I would also like to thank my family and friends, without whose support and encouragement, I would not have finished this thesis.

Contents

List of Tables

List of Figures

Abbreviation

BSH	Bundesamt für Seeschifffahrt und Hydrographie
WKP	Wiederkehrende Prüfungen
RCM	Reliability centered maintenance
RBI	Risk-based inspection
BetrSichV	Betriebssicherheitsverordnung
SeeAnV	Seeanlagenverordnung
SOLAS	Safety of life at sea
IMO	International Maritime Organisation
PI	Periodic inspection
PoF	Probability of occurrence of failure
CoF	Consequence of failure
FLS	Fatigue limit state
RCR	Redundancy ratio
ALS	Accidental limit state
2D/3D	two-/ three dimensional
ULS	Ultimate limit state
SCF	Stress concentration factor
GS	Geometric stress
GSR	Geometric stress range
IIW	International Institute of Welding
FEA	Finite element analysis
FEM	Finite element method
API	American Petroleum Institute
LAT	Lowest astronomical tide
SBI	Safety based inspection
HAZID	Hazard identification
GDV	General Association of the German Insurance Industry
IP	Ingress protection
HVAC	Heating, ventilating and air conditioning
MSC	Maritime Safety Committee
Cir.	Circular
P&ID	Piping and instrumenting diagram

1 Introduction

> *"Most people become engineers because they feel at least some affinity for things, be they mechanical, electrical or structural. This leads them to derive pleasure from assets in good condition, but feel offended by assets in poor condition."* – John Moubray.

In the last decades, the human being has to confront with the energy crisis and climate changes. From these hard situations the heavy research, development and investment of the renewables energy industry arose as a primary solution. Regenerative energy sources, which do not exploit Mother Nature, supply the sustainable energies and enhance the standard of living, are becoming the solutions for the future and the centre of energy-related issues. But nevertheless, it is impossible to use any natural sources without affecting their qualities and the broad-ranging interdependencies. This has raised a new challenge for engineering.

In Germany, offshore wind energy ranks as the main objective of the policy of energy transition. The Federal Maritime and Hydrographic Agency (Bundesamt für Seeschifffahrt und Hydrographie – BSH) is the German authority who issues operation permit for offshore installations with consideration of all the project phases including development, design, execution, commissioning, operation, life time extension, and dismantling. The licensing procedure is structured correspondingly to these phases, where a competent independent third party is engaged as surveyor or certifier in every stage. A time schedule for these phases in sequence is given in [1], included herein as appendix 01.

In the operation phase, to maintain the validity of an operation permit, the BSH requires periodic inspections to be carried out for offshore wind power stations and substations in places within the German exclusive economic zone.

> *"Durch Wiederkehrende Prüfungen (WKP) ist der Zustand der Offshore-Bauwerke in der Betriebsphase zu überwachen. WKP sind zur Aufrechterhaltung der Betriebserlaubnis erforderlich."* (Standards BSH [1], P.20)

Beyond the legal force of consent, periodic inspections aim for proper operation, which ensures the safety of personnel, assets and the environment during the service life of the installation. Furthermore especially on a manned platform, safety of personnel shall be guaranteed for all the time. This can be accomplished by an elaborated maintenance and surveillance program.

A fixed offshore substation consists of three main parts, namely the foundation element – called piles, supporting structure - called jacket, and the facilities of operation -

called topside. Figure 1-1 shows an ordinary construction of offshore stations with indications of sub-constructions.

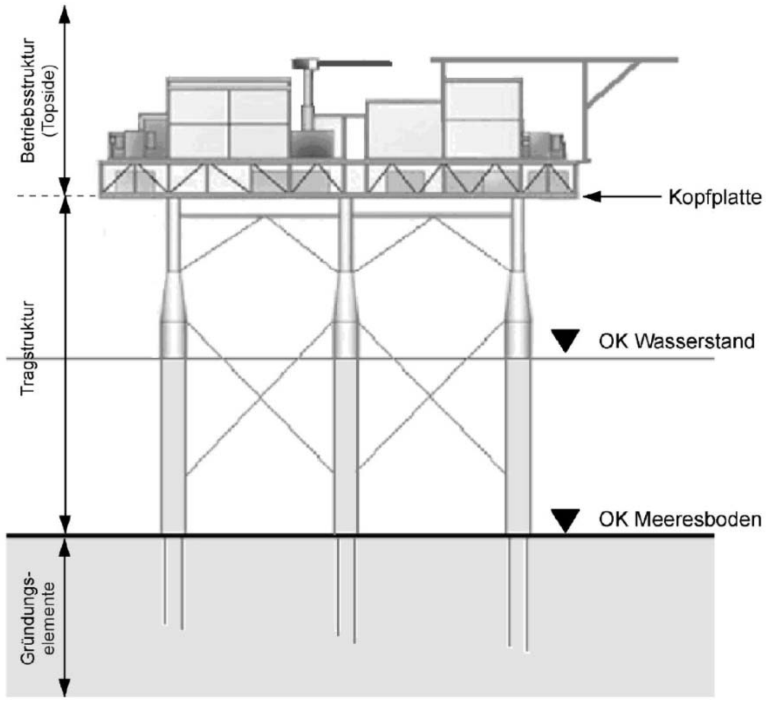


Figure 1-1: Construction of an offshore station, source [1] P.20

Periodic inspections of these parts differ from each other in essential aspects - from technical expertise, technique, to organisation and assignment of responsibilities. The planning of inspections of each system has to be adjusted appropriately.

In this thesis, issues concerning piles are not considered due to specialized knowledge necessary. For structural and plant systems, Table 2.1-1 features characteristic differences to be considered in the system maintenance. The planning of inspection of each system requires therefore different handlings concerning inspection objects, methods, test types and also intervals. Most notably, a structural system, in this case an offshore jacket, behaves as one body, while a plant system is assembled of various components which undertake different functions.

Table 2.1-1: Structural and plant systems

References	Structural system	Plant system
Objective	Structural integrity	Proper functionality
Subject matter	Steel structure	Mechanical components
Function	Load carrying	Diverse
Operating condition	Underwater, wave	Offshore atmosphere
Hazard	Wave load, accidental loads…	Corrosion, ignition, pressure, human errors…
Failure	Cracks, Buckling…	Leakage, malfunctions…
Accessibility	Restricted	Extendable

The similarity of these two systems is in the complexity and limited possibility for inspection within a reasonable scope.

An offshore jacket is a framed construction employed under water. It is impossible to inspect the whole system without major financial efforts and putting the diving crew at risk. It depends on the accessibility of particular areas of the structure with regards to structural obstacles or marine growth. Although the marine growth can be removed, it demands activities under water and again financial efforts and danger. The question is: Is it really necessary to check all parts of a jacket at that considerable cost or can the scope of work be limited in a reasonable extent and still capture expressively the condition of a jacket?

The same question is also proposed for planning periodic inspections of a plant system. Apart from a correct maintenance process, what should be done in a periodic inspection to acknowledge the operational readiness and the designated functioning, without exceeding available capabilities? It is to bear in mind that a plant system consists of a series of components which differ in design, have different functions and thus demand different test and professional expertise. The goal of the condition monitoring is to assure the reliability of each component and their interactions, so that they can perform properly. Especially the safety-relevant systems of the topside, which can have an immediate impact on people, shall be fully and continuously operable.

Considering this matter, it is suggested to define the scope of periodic inspections with a risk- based method, which is a selective approach with clearly defined criteria. In this way, the scope can be confined rationally and meaningfully. A failure is first defined with regards to object and purpose of periodic inspection. The probability of occurrence and consequence of the failure are then defined in terms of appropriate parameters. Following, risk of the failure can be evaluated.

This thesis deals with a simplified risk-based approach for planning periodic inspections of an offshore converter platform. In the following chapter, chapter 2, an overview of periodic inspections are given and the standards of technical issues which are used herein are also introduced. The methodology is presented and illustrated by analysing an offshore jacket with the FE-model established by the department Offshore Installations of the company DNVGL in chapter 3, and a sample water fire extinguishing system usually surveyed by the department Plants and Pipelines in chapter 4. In general, a system failure is first defined and assumed. Risk in terms of the probability of occurrence and the consequence of this failure is assessed. Thereby definitions of these two risk parameter are also clearly stated with regards to the characteristic qualities of the concerned system or the hazard to which the system is exposed.

In chapter 5, a conclusion is given which summaries the methods and their implementations described in the prior chapters. At last a remark is provided for the application of risk-based methods in the practice and the prospect for a rational solution in the future.

2 Periodic Inspections

2.1 General

Periodic inspections are a part of preventive maintenance for technical systems, carried out based on predefined intervals without awareness of damage or failure of system. Periodic inspections aim for monitoring the latest condition of the system with regard to degradation over the service life, updating the database for evaluation and adjusting the maintenance program. Any information from the last inspection will be used for analysing system integrity, assessing effectiveness of the protection, or initiating of corrective measures if required. It demands cost-intensive efforts of staff and technology as well as testing work for an appropriate inspection. Figure 2-1 shows the general modularity for maintenance of a technical system, in which the principle of total or selective perception can be integrated.

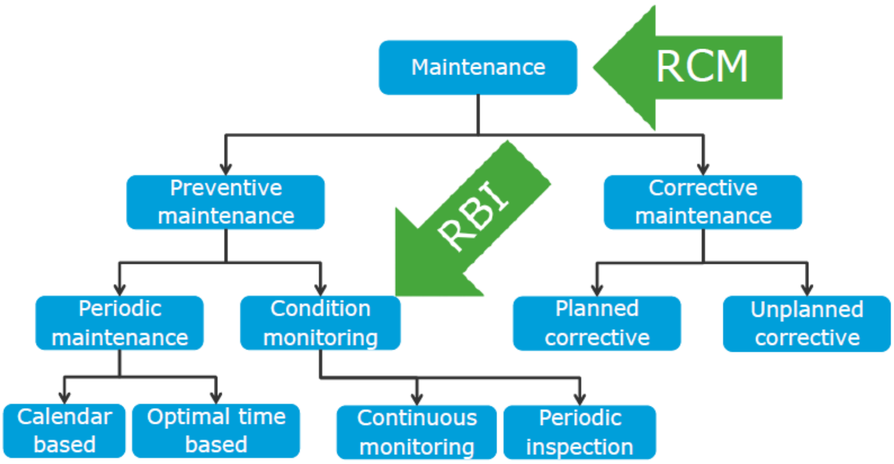


Figure 2-1: Scheme of maintenance activities, source [6] P.7

Although the periodic inspection is part of the maintenance regime, it shall be performed by an independent third party, who is accredited for concerned expertise, according to BSHs Standard [1]

> *"Wiederkehrende Prüfungen im Sinne dieses Standards sind regemäßige Prüfungen der Komponenten eines Offshore-Bauwerks durch den Prüfbeauftragten/Prüfsachverständigen. Sie dienen zur Feststellung und Beurteilung des Istzustandes. Die wiederkehrenden Prüfungen erfolgen entsprechend einem durch Betreiber aufgestellten und mit dem Prüfbeauftragten abgestimmten Prüf- und Inspektionsplan."* (BSHs Standard [1], P.139)

As technical systems grow and undertake more and more complex commissions, their maintenance confronts an extended duty and challenges. In the late 70s, the general definition of maintenance, *"ensuring that physical assets continue to do what their users want them to do"* ([17], P.7), has been enhanced and specified to one of the new concept of maintenance – Reliability Centered Maintenance (RCM). RCM is defined as a *"process used to determine what must be done to ensure that any physical asset continues to do what its users want it to do in its present operating contexts"* ([17], P.7). The main idea of RCM is to investigate the asset under operating conditions in terms of: performance standard, failure modes, failure causes, failure effects, failure consequences, counter measures (including predictive, preventive and corrective) and alternative corrective measures. It aims for a purposeful and effective maintenance strategy.

As seen in Figure 2-1, periodic inspections are a part of condition monitoring and not to be confused with periodic maintenance. Because periodic inspections are actually carried out without awareness of failure, its scope may be as large as the system itself. In the concept of reliability centered maintenance (RCM), the risk based inspection (RBI) is considered an essential means which helps identifying the hazardous parts or areas, to focus on them and result in repair or determination of intervals and intensity of the inspection or further measures. Instead of inspecting the whole system, RBI selects only components exposed to high risk and adjusts the inspection plan for them accordingly. With this method, problems encountered in the operation for a large engineering system can be identified in advanced, classified and prioritized for the inspection process. This enables realistic human and technology efforts and makes it possible to use the budget cost-effectively.

Risk in RBI is assessed based on the relation of components to the system, or the potential hazard to the system, and how probably a failure of system may occur. RBI of a structural system and a plant system differ from each other in application of the methodology and definition of the criteria. Strategies for planning periodic inspection are detailed in chapter 3 and 4. Following chapter is about the discussion of applying standards and rules for the inspection.

2.2 Standards and Guidelines

The BSHs Standard for Construction – "Minimum requirements for construction of the offshore substations in the German exclusive economic zone" (28.07.2015) - was released at the end of 2015 as the further elaboration of the BSHs Standard – "Design of offshore wind turbine" (2007). Intermediate results of this development are draft versions dated from 19.09.2014 and 29.05.2015. The updated standard includes not only the offshore wind turbine but also cabling within the wind farm, measurement mast, offshore station and power-cable system.

According to BSH, offshore installations and employed systems shall be compliant with regulations and standards recognized by BSH, whereas the European rules and guidelines shall be applied primarily. For items, which are not designated in Eurocodes, other international and national recognized standards are to be applied.

> *"Es ist nach den Eurocodes vorzugehen, dazu sind die Normen des DIN in der jeweils jüngsten als Weißdruck veröffentlichen Fassung anzuwenden… Abweichungen von den hier geregelten Vorgaben und Anforderungen sind möglich, soweit diese aufgrund neuerer Erkenntnisse allgemein oder aufgrund der vorhabenspezifischen Besonderheiten erforderlich oder den genannten Schutzzwecken in mindestens gleichwertiger Weise zu dienen geeignet sind."* (BSHs Standard [1], P.17)

Eurocode is the general designation of the norm series DIN EN 199x with the national appendixes and supplementary rules referred in the latest publication (BSHs Standard [1], P.129). Generally Eurocodes are developed for civil engineering. However offshore conditions are very different and have therefore specific requirements. In the course of planning inspections for offshore station, these requirements shall be taken into account to constantly ensure the safety first and foremost, and secondly the fitness for purpose of the entire plant. Offshore specific requirements and their satisfaction have been studied and standardized in the oil and gas industry as well as in the maritime industry. Their applicability for offshore station can be taken into account.

2.2.1 Standards for Offshore Support Structures

Apart from the Eurocodes, other regulations are also recommended in the BSHs Standard, such as NORSOK, DNV-rules, GL-rules, etc. For issues, which are not regulated in these standards, further recognized standards can be applied. Higher requirements are permitted.

> *"Die in den jeweiligen Kapiteln genannten Normen sind durch weitere Regelwerke, Richtlinien und Empfehlungen zu ergänzen, soweit sie Aspekte der Bemessungen von Gründungselementen und Tragstrukturen für Offshore-Bauwerke nicht berücksichtigen."* (BSHs Standard [1], P17)

> *"Höhere Anforderungen in Bemessungen und Ausführung sind teilweise üblich und ausdrücklich zulässig."* (BSHs Standard [1], P116)

ISO 19902 is an international standard for fixed offshore structure in the oil and gas industry. An offshore platform, whether a drilling rig or a power station, consists of a jacket support structure and topside for the purpose of processing of petroleum or conversion and transformation of electricity. It bears an analogy with regard to working condition, functional and constructional design. ISO 19902 may be thus applied in principle.

To approve the suitability of ISO 19902, the company DNVGL has established a comparative study between Eurocodes and ISO 19902 regarding extreme limit states of the same offshore jacket, which claims the limitation of Eurocodes and by comparison the satisfaction of higher requirements of ISO 19902.

> *"Mit der ISO 19902 werden konsevativere Ergebnisse für die Auslastung an den Rohranschlüssen und Rohrelementen berechnet."* (DNVGL Report [3], P.4)

Here the strength of tubular connections is considered as the criteria for strength verification for offshore structure. This is because of the three-dimensional configuration of welded tubes under high lateral loads. In this situation it covers especially the phenomenon of punching through, which has a significant impact to the structural integrity. A basic step needed for verifying structural integrity is to check the resistance of the structural subcomponent, tubular member and tubular joints. Beside this, another difference between Eurocodes and ISO 19902 is the degree of detailing in type classification of tubular joints. Types of connections are distinguished acc. to EN 1993-1-8 by the geometry, whereas acc. to ISO 19902 both geometry and force flow in the connections are considered. A thorough joint type classification contributes to the accurate calculation of existing stress in the material. Furthermore, not only in-plane but also out-plane bending loads are specified in ISO 19902. In the aforementioned comparison study, it came to a conclusion that the typical tubular joints used for offshore structure are not included in the application field of Eurocodes.

> *"Abschließend kann festgelegt werden, dass die typischen Rohranschlüsse für Offshore-Strukturen nicht vom Anwendungsbereich des Eurocodes abgedeckt sind…"* (DNVGL Report [3], P.2)

In this thesis, ISO 19902 is applied further in the calculation of utilization of tubular connections, which is used as the criteria for evaluation of structural integrity and a part of risk assessment.

2.2.2 Standards for Auxiliary Systems on Offshore Platforms

For an auxiliary system on a platform, for instance a cooling system or an extinguishing system, it is difficult to point out a particular rule to apply for periodic inspection. Such a plant system is composed of many autarkic components, which feature their own function and cooperate with each other through the measurement, control and regulation technology. Each component is therefore compliant with a specific regulation and requires a corresponding inspection plan. They are usually type-approved and tests are to be carried out by an expert in the particular technical field. Therefore maintenance activities shall be elaborately planed. They should at least cover operational requirements by the manufacturer. But in most cases, the products are reliable if the operational conditions are as defined for these products and the operation manuals are properly kept up. For a fixed converter platform, it may cause difficulties for the operator concerning these two matters. Offshore corrosive conditions demands specific technical coherence and put the installations in need of care. The system is considered reliable, if the offshore requirements are additionally satisfied. However, these are minimum requirements aiming at safety, given in such offshore standards as "Offshore substation for wind farms – DNV-OS-J201 ([4])". For a proper operation, higher requirements are expected.

From the certifier's point of view, an overall and comprehensive inspection shall be performed, which is able to report the general status and operational readiness of the system. PIs are to be performed under the condition that the maintenance is accomplished according to operational manuals.

A water fire extinguishing system is a safety-relevant system. It means that the system's functioning is the equivalent to platform's safety and shall be guaranteed at all times. Therefore the PI for fire extinguishing system is to be carried out to confirm the reliability of the system. Along with it, the fulfilment of requirements for safety and health shall be approved. Referred hereto are all encompassing regulations such as Ordinance for Industrial Safety (Betriebssicherheitsverordnung- BetrSichV, Offshore Installation Ordinance (Seeanlagenverordnung – SeeAnV), recommends for safety of life at sea (SOLAS), International Maritime Organisation (IMO), etc. This is clarified further in the chapter 4 of this thesis.

2.2.3 List of Standards

The standards referred herein are given in the list of literatures at the end of this paper. Following is an extraction from the list to highlight the requirements to be complied with.

[1] Standards Bundesamt für Seeschifffahrt und Hydrographie 2015, *Mindestanforderungen an die konstruktive Ausführung von Offshore-Bauwerken in der ausschließlichen Wirtschaftszone (AWZ)*, BSH Standard Konstruktion, [28.07.2015].

[2] Standards International 2007, *Petroleum and Natural Gas Industry – Fixed steel offshore structure,* ISO 19902:2007, International Organization for Standardization, ISO.

[3] DNV GL AS, Oil & Gas, Offshore Installations 2015, *Vergleichsstudie zwischen BSH und ISO 19902, Report nr. GLO-15-794,* DNVGL AS Report. Available from: DNV GL, Oil & Gas, Offshore Installations, [11.06.2015].

[4] Det Norske Veritas 2009, *Offshore Substations for Wind Farms*, DNV OS J201, Standards Det Norske Veritas, DNV.

[16] International Maritime Organisation 2012*, Revised guidelines for the maintenance and inspection of fire protection systems and appliances*, MSC.1/Circ.1432, IMO.

3 Strategies for Periodic Inspections of Offshore Jacket Structures acc. to ISO 19902

3.1 Risk-Based Inspection

3.1.1 General

The major objective of periodic inspection is the detection of deterioration of the system over time. It refers to many aspects, among others, corrosion, cathodic protection, or also scour development around piles at seabed. Condition monitoring regarding these matters demands frequently inspections. The abrasion of coating is predictable and measurable, so that the intervals can be calculated correspondingly. And although scour influences the structural integrity, it is assumed to have sound condition concerning scour in the process of risk assessment. In case of existing scour, the overall integrity shall be checked comprehensively, including bearing capacity of piles and overturning moment. Periodic inspections (PIs) of these conditions are not considered in this paper.

PIs herein refer to the structural degradation of the jacket structure, which impacts the load carrying function in a long-term view, the fatigue fracture. To define the scope of PIs, potential failures are first to be identified and localized. Prognoses and expectations of failure can be made by means of theoretical structural investigation before they occur. Through this predictive process, the scope of PIs can be confined and carried out as a target-oriented preventive measure.

Why is fatigue failure the central object for PIs of offshore jacket? We have agreed that PIs are for condition monitoring, for the degradation of the designated capacity or ability. Waves possess massive energy and exert large varying lateral forces on the slender structure of the jacket. On top of this, an offshore jacket is composed of welded tubular beams in sequence and in three dimensions. They exhibit a high sensitivity to fatigue and are therefore the subject matter of the risk assessment.

An offshore jacket is to be designed to withstand all the conceivable loads in offshore condition. Strength verification shall be made for transporting situations, for in-place situations, for accidental situations and also for seismic situations, if applicable. However, they are done with the assumption that the designed structure is in an intact condition. A damaged condition can only be considered and simplified by the safety factors used in the design calculation. This chapter deals with such an offshore jacket as a finite-element model, which is verified for design. The jacket researched herein is situated in the operation phase, subjected to extreme environmental action in a fatigue-damaged condition.

In a fatigue analysis, fatigue critical spots of the structure can be identified to provide a concrete damaged condition. This damaged condition is defined by the failing of a structural element, which is directly linked to such a particular fatigue critical spot. Probability of occurrence and consequence of this damage on the overall integrity are parameters for risk assessment of this fatigue failure.

Table 3.1-1: Risk matrix

Probability of Failure					
	high-4	4	8	12	16
	m.high-3	3	6	9	12
	m.low-2	2	2	6	8
	low-1	1	2	3	4
		low-1	m.low-2	m.high-3	high-4
	Consequence of Failure				

Risk = Probability of occurrence x Consequence of failure

Table 3.1-1 graphically presents the definition of risk as the product of occurrence probability of an in-place fatigue failure (PoF) multiplied with its consequence (CoF) on the structure in in-place situation. Areas of high risk level shall be prioritized for PIs in terms of frequency and intensity of inspection. This method makes it possible to concentrate at suspected areas and thus to perform the inspection properly. Risk categorizing in this manner facilitates adjusting the scope of PIs to available capacity and technical provision.

3.1.2 Qualitative and Quantitative Risk-Based Inspections

Determining the risk parameters can be performed qualitatively or quantitatively. In other words, assessment of PoF and CoF can be executed empirically or numerically.

The qualitative approach the RBI requires much of experience and expert opinion. Critical spots are identified based on gained knowledge, repeated failure patterns or laboratory results. A default inspection program is given in ISO 19902-claus 23.7.

In contrary, the quantitative approach, as per the DNVGL Report [6], contains detailed calculations of probability of a fatigue failure. The probabilistic models used are based on the structural reliability analysis methods, where uncertainty of any event, such as the real fatigue lifetime or the real resistance of the structure, is statistically considered and factored in the probability of occurrence of a particular failure. For example, Figure 3-1 from the DNVGL report [6] display a SN-data and curve with the statistical uncertainness of the fatigue lifetime for a particular stress.

> *"An example of the input to a probabilistic model is the fatigue capacity expressed using SN-curve. SN-curves are based on experimental data and therefore contain natural variability of the fatigue capacity, reference made to Figure 3-8 (Figure 3-1 herein). The variability inherent in the SN-curve is modelled in a probabilistic/limit state model which is used in a quantitative risk based inspection (RBI) plan."* (DNVGL Report [6], P.13)

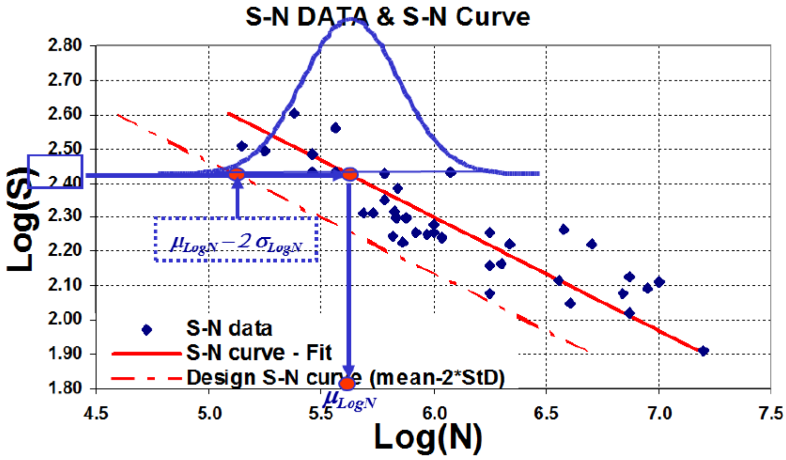


Figure 3-1: SN-data and resulting design and characteristic SN-curve, source [6] P.13

The SN- data is displayed in a double logarithmic plot, so that the mean values are akin to a curve. The uncertainty variable in this stochastic process is the fatigue lifetime which may come out from the experiments of specimens exposed to a particular stress. In this case, it is a normal distribution and the probability density function has a bell-curve form. Mean values are fatigue capacity with the highest probability of appearance but they are not used for design. The values taken for design in this example are the lower values, twice of the standard deviation from the mean values. The design curve is the SN-curve fit in Figure 3-1. Thus the corresponding probabilities of design values are taken into account in the calculation of the probability of fatigue failure for the quantitative RBI.

Further probabilistic models are also considered in a comprehensive quantitative RBI. They are, for instance, with stress range as uncertainty variable in a Weibull-distribution, or with safety factor (behaviour of carrying capacity to loading effect) as uncertainty variable in a probability distribution determined using the first order reliability method. With this approach, probability of failure can be explicitly numerically calculated. It relies heavily on analytical modelling and computational evaluations. The result is transparent, traceable and offers base data for further analysis.

The method used in this thesis is not an absolutely qualitative approach and also not a quantitative approach. It is a semi-quantitative approach to RBI, which is a compromise to take advantages of both and constraint the disadvantages. The Pof and CoF are not accurately calculated but assigned in levels as in Table 3.1-1.

The PoF in this method is not the probability of occurrence in the conventional understanding of statistic. The failure is therefore not a random variable but a certain event which happens when the fatigue life is over. The probability of occurrence in this sense is therefore not how likely the failure would occurs, but how early it would occurs. If the potential failure can be avoided, by grinding for example, the fatigue capacity is restored. In a fictive long-term view, a structural element with short fatigue lifetime would come to failure repeatedly often, while an element with long fatigue lifetime more rarely. PoF is therefore not evaluated for one single structural element but for all elements of the structure and sorted into PoF-levels due to their fatigue lifetimes.

The consequence of fatigue failure with this method is evaluated by calculating the structural resistance of the jacket to the loading effect, thereby random variation of these values is not considered. Table 3.1-2 shows the key differences between a quantitative approach (right column) and the introduced semi-quantitative approach (left column).

Table 3.1-2: Semi-quantitative and quantitative approaches

	Semi-quatitative	Quantitative
Objective	Fatigue failure	Fatigue failure
Risk =	PoF x CoF	PoF-fatigue x PoF-collapse
Fatigue failure model	Cumulative damage	Cumulative damage and Cumulative probability of failure
PoF	Fatigue lives in comparison	Probability of occurrence
Probabilistic models	PoF: Frequency of the same failure in a fictive long term observation	PoF-fatigue: Probability density function with uncertainty variables: material, geometry, loading. (using Weibull- statistical theory for material uncertainty, etc.)
Probabilistic models	-	PoF-collapse: Probability density function with uncertainty variables: resistance and stress (using the first order reliability method)
Consequence of failure	RCR Criterion: member or joint failure	RSR Criterion: system collapse

In a quantitative approach, CoF is determined as the reserve strength ratio (RSR) of the structure in damaged condition, also defined in ISO 19902 –[2], involves an unfactored environmental action which causes collapse of the structure. This requires very comprehensive structural analysis and provides results in details. To simplify, a redundancy ratio is analogously defined, which does not include the environmental action causing collapse of the whole structure but causing the exhaustion of the structural element-capacity. This semi-quantitative approach is further clarified in following chapters.

3.1.3 Semi-quantitative Risk-Based Approach

Generally PoF can be determined first or CoF first. But in the process of fatigue analysing to identify the weak spots in the service life, the PoF is also intimated. The PoFs of all structural elements are therefore to be determined. Based on this result, the CoF of elements with high PoF will be then evaluated.

Fatigue capacity of a structural element, a tubular member or a tubular joint, is implied in the fatigue lifetime, which can be calculated directly from the cumulative damage in a particular period of time. The greater the damage, the shorter the fatigue lifetime and thus the lower the fatigue capacity is.

The evaluation of CoF is then based on the assumption that the failure has occurred. CoF in this case is the reduction of the structural capacity of structural member and structural joint, which can more or less represent the structural capacity of the jacket. Element- capacity is therefore used as reference for CoF because in case of an element failure, it leads to load redistribution over other elements. All structural elements are dimensioned for load carrying, of which the contribution creates the rigidity of the structural as a whole. When local loading exceeds the calculated resistance, new failure may occur, the load redistribution repeats, and the process continues. In worst case it may end in collapse. It is therefore essential to figure out the most probable failure.

3.1.3.1 Probability of (occurrence of) failure

As mentioned in 3.1.2, the PoF in this approach is not determined in a stochastic process. The PoF here is not understood as likeliness of a failure event but as the degree of fatigue capacity. Instead of a detailed calculation of probability of each single failure in percent in a quantitative RBI, PoF in this sense is determined for all joints in a fatigue limit states analysis (FLS). A computational program enables to calculate the fatigue limits of individual structural components. Ranking fatigue lifetimes of components demonstrates the degree of reliability of components against fatigue. In this simplified method, the SN-curve used is the design curve given in ISO 19902 [2] without consideration of the random variable of the fatigue capacity and of the loading. The designed values are used for calculation of the cumulative damages of all tubular joints of the structure (see chapter 3.2) for a period of time. These damages are then compared with the acceptable values for this period and assigned accordingly into PoF-levels. The period of time chosen is usually the lifetime of the jacket, so that the damage of 1,0 represents the exhaustion of fatigue capacity and the failure is initiated at this element.

The attribution of damages into PoF-levels is decided by the results of FLS concerning severity of the calculated damages and the amount of elements with significant damages, and available inspection performance. For instance, the operator chooses 25% of the elements, which have the highest damage to undertake CoF-evaluation for them. If these CoFs are acceptable, more elements can be chosen to be subjected to CoF-evaluation. This proceeds until the inspection ability is utilized or the scope according to regulatory requirements is adjusted. Table 3.1-3 shows an example for the attribution of cumulative damages after a design lifetime of the jacket into PoF-levels.

Table 3.1-3: Level of PoFs

PoF-level	**1**	**2**	**3**	**4**
Cummulative damage	0 – 0,3	0,3 – 0,5	0,5 – 0,8	>0,8

With this method, the PoFs can be calculated for all components of the jacket. The relation between their damages gives us an overview of fatigue resistance of the structure in in-place condition for the intended period. It provides a direction and basis for further analysis and countermeasures if required.

In an offshore jacket, structural components are tubular elements and tubular joints. Tubular elements are tubes welded together in sequence to build braces or chords of the jacket. Tubular joints are tubes welded together angularly to build connections of beams or connections of beams and the main legs of the jacket. One joint, in a FLS, consists of two components, a brace and a chord. In a joint, when the end of one beam is welded to the body of the other beam, the beam with the end welded is called brace and the beam with the body welded on is called the chord. The calculated damages always refer to the weld seam of a joint. If the joint is selected for evaluation of CoF, failure is assumed as occurred and the brace is assumed as teared off from the chord. The tearing-off brace is declared as non-structural. CoF is defined as the consequence of this event on the structural integrity and evaluated as in 3.1.3.2.

3.1.3.2 Consequence of failure

Actually, CoF of a technical system is assessed in the three perspectives of consequence: negative impacts on personnel, on asset and on the environment. An offshore jacket, which supports the platform above water level, is constantly exposed to wave and current forces. This carrying function of the jacket is the first requisition for such a fixed offshore installation. It is the existential condition for the facilities as well as the operation. System collapse would lead to loss of life, and water contamination due to operating substances. In this context, the CoF in terms of structural deterioration implies also the CoF in terms of consequences that are important for safety and for the environment. The CoF is therefore evaluated by structural analyses and demonstrated as structural key values. The calculation process is presented closer in chapter 3.2.

Based on the results of PoFs, the necessary analyses for CoF are undertaken for each component with high PoF determined as in 3.1.3.1. The extent of the selection depends on the intermediate results of CoFs. The evaluation shall proceed for one joint to another until the amount of joints to be inspected or joints at a definite risk level is reached.

As mentioned above, the CoF is the consequence of the loss of the damaged brace. This is to be removed from the model of the jacket to simulate the jacket in damaged condition. Structural analyses are to be carried out on the damaged jacket to ascertain severity of the failure. In a quantitative approach, severity of the failure or CoF is assessed by the reduction of the jacket resistance against collapse. In the simplified method, CoF is evaluated by the reduction of the jacket resistance against utilization of member and joints. A redundancy ratio (RCR) is therefore defined to represent the jacket resistance against component utilization.

$$\mathrm{RCR} = \frac{F_{fc}}{F_{100}} \qquad 3.1\text{-}1$$

F_{fc}: is the un-factored global environmental action which, when co-existing un-factored permanent and variable actions are added, where member or joint is utilized to capacity.

F_{100}: is the un-factored 100 year global environmental action calculated in acc. to ISO 19902-clause 9

The RCR refers to the capability of the jacket to withstand the abnormal environmental action, while the jacket can still conserve its intactness and be in the sound condition. The utilization of member or joint is used here as the criterion of structural integrity of the jacket as mentioned in 2.2.1. When the designed strength or capacity of a member or joint is exceeded, equals the utilization over 1,0, plastic deformation will occur or in worst case the load carrying function will completely fail. It is the failure of one element but leads to the load redistribution over the whole structure. That's why RCR can represent the structural integrity and be used for the classification of CoF.

The reduction of RCR is evaluated on a percentage basis as follows:

$$a = \frac{RCR_0 - RCR_1}{RCR_0} \cdot 100 \qquad 3.1\text{-}2$$

RCR_0: Redundancy ratio of the original jacket

RCR_1: Redundancy ratio of the damaged jacket

The attribution of CoF to levels is done on a percentage basis, and again, depending on the severity of the consequence and the framework of the inspection. Table 3.1-4 show a sample classification of "a" (as in the formal 3.1-2).

Table 3.1-4: Level of CoFs

CoF-level	1	2	3	4
Reduction of RCR	0-20 %	21-40%	41-70%	71-100%

However the classification of the "a" shall be decided by the operator in consideration of many other factors. It can be the firm philosophy, specific safety requirements or the limit of budget. This can also vary over time. The older the structure becomes, the more will be invested for inspection. When the CoF and the associated PoF have been assessed, components under significant risk can be identified and chosen to be subjected to the PI.

3.2 Risk Assessment

3.2.1 The Assessment in Work Steps

This chapter contains the theoretical basis for calculation of the related structural key values, according to ISO 19902 ([2]), and thus the classification of PoF and CoF as defined in the previous chapters. Figure 3-2 displays the calculation process in a flowchart to visualize the work flow of risk assessment, or risk parameters assessment. The following subchapters explain closer the work steps and structural analyses. However, the central subject matter of the risk assessment is the fatigue failure. Therefore the fatigue analysis emphasized. Other analyses are only abstractly presented as reference for the calculation.

At the beginning of the operation phase, a baseline inspection is to be carried out to confirm the intactness of the jacket after the manufacturing, transport and installation. If there is no finding, the employed jacket is seen satisfying all design requirements. The model 0 in Figure 3-2 represents such a jacket. Model 0 is the original jacket, fixed to the seabed and exposed to the weight of the topside, to the wind, wave and current loads.

Model 0 is first subjected to the fatigue limit states (FLS) for a period of the designed lifetime. The results are calculated cumulative damages of each single joint of the jacket. The FLS considers only the tubular joints because weld seam at tubular joints are more critical than those at the tubular members. Having the damages of each joint, it can be assigned into the appropriate level of PoF as in 3.1.3.1.

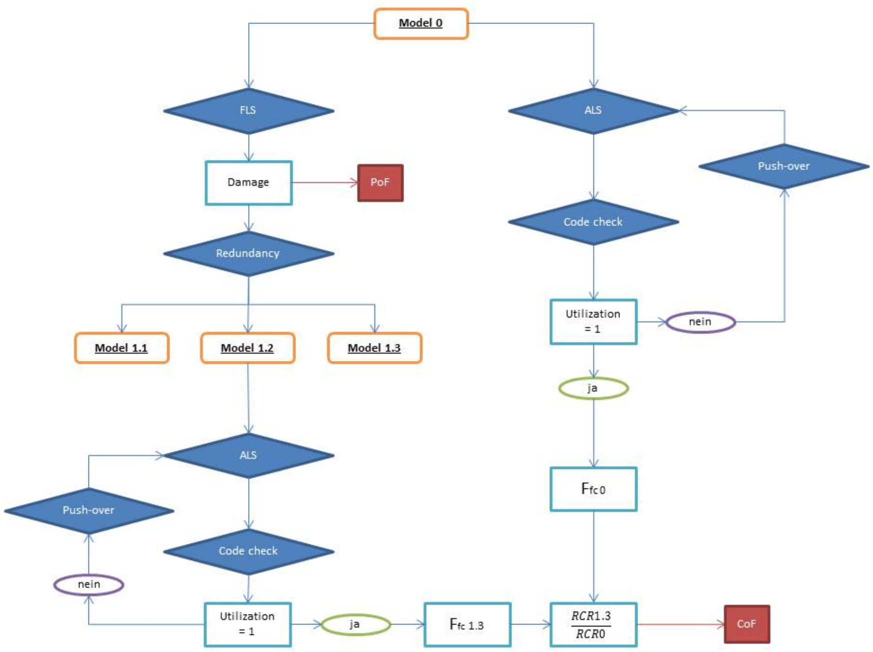


Figure 3-2: Workflow of risk assessment for structural system

The jacket with critical spots, or with the joints identified as having high PoF, is then subjected to the redundancy analysis. The redundancy analysis herein is an analysis in which the object is modified and subjected to further analyses to investigate the impact of the concerned modification. It refers in this case to the removing activity of a brace from the jacket to create the damaged jacket, or the jacket in damaged condition (Model 1.1, Model 1.2 or Model 1.3). The selected brace is the one involved in a chosen joint with high PoF. The index number 1.2 refers to the jacket originated from the Model 0, with fatigue failure in first order, and with the brace from joint 2 removed. A fatigue failure in second order would be the fatigue damages calculated on the revised jacket Model 1.1, 1.2 or 1.3. Fatigue failures in second order are not considered in this paper.

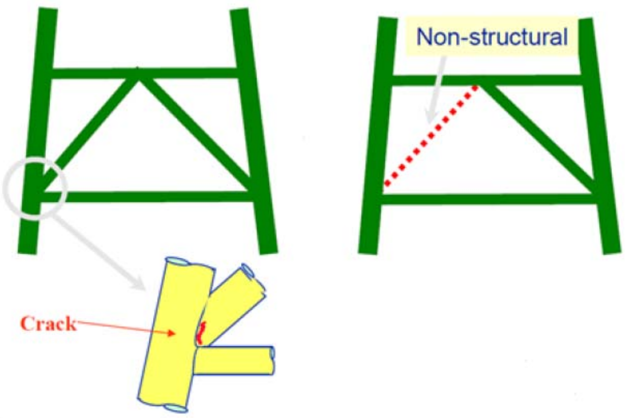


Figure 3-3: Example Model 0 (left) and Model 1.2 (right), source [6] P.21

The damaged jacket is then subjected to a series of structural analyses to determine the CoF of the fatigue failure assumed. The RCRs as in 3.1.3.2 of the original intact jacket (Model 0) and of the damaged jacket (Model 1.2) are to be calculated. Because both models are equally subjected to the same procedure, they are only called “models” in the following description.

As defined in 3.1.3.2, the RCR is the ratio of the F_{fc} to F_{100}. The F_{100} is the 100-year un-factored environmental action. This is assumed herein as the load derived from the highest wave with the returning period of 100 years. According to weather report for the region of the North Sea, it can be up to 17m height. The F_{fc} is an un-factored fictive environmental action, at which the utilization of member or joint reaches unity – 1,0.

In order to determine the F_{fc}, the model is subjected to the accidental limit state analysis (ALS). In the ALS, the jacket is exposed to extreme values with respect to loads and modification of both partial load factors and partial resistance factors. Objective of ALS is to investigate the jacket in damaged condition without any safety factor. The analysis expresses the case, in which the conceivable uncertainties may occur. The utilization of member or joint is to be calculated within the ALS.

Because the requirements for design of an offshore jacket are relative high and satisfy many structural requisitions, the utilization of member or joint under the load of F_{100} is usually under 1,0. In the full-capacity analysis, the environmental load is to be increased until the first utilization of 1.0 is reached. It is the force of full capacity F_{fc}. However, it is a simplified full-capacity analysis which takes the base shear into account under the condition of the ALS.

3.2.2 Quasi Static Analysis

For FLS or ALS, environmental loads are assumed as quasi static. The loads on the structure can be calculated in terms of the momentary velocity and acceleration of the wave and current. The wind and the current are assumed to have a constant velocity and the same direction with the wave. The wind exerts forces on the topside, so it can be modelled as point load on the topside. Wave and current loads can be calculated according to ISO 19902- chapter 9.5.

This chapter summarizes the procedure of calculating the load derived from wave and current on a jacket structure. It encompasses also assumptions as well as the mathematic basis of each step. Figure 3-4 display the workflow in a block diagram according to ISO 19902 ([2]). It describes the transforming of wave hydrodynamic into quasi static loads which are useable for computing load effects generated on the jacket. The calculation process encompasses many observation based assumption, mathematical analyses as well as empirical factors. However, its plausibility and reliability is proven by the successful outcome over many years.

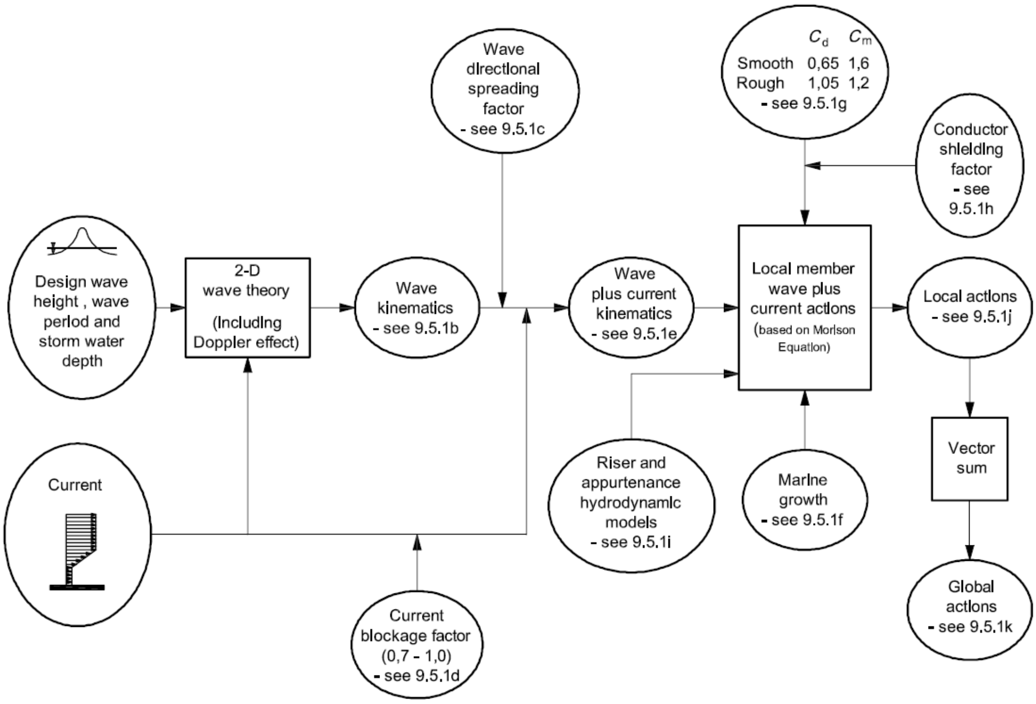


Figure 3-4: Procedure of calculating the quasi-static action caused by wave plus current acc. to ISO 19902, source [2] (P.53)

The procedure is applied to calculate loads caused by one wave with its characteristic height and period. With means of wave theories, wave kinematic can be derived from these physical properties. The results are demonstrated as a 2D-vector field, and applied equally in 3D- space to simulate the ambient actions on the jacket. With the known kinematics with their specific coordination and the geometry of the jacket, the loads exerted on the jacket can be calculated. This is done by interpreting the situation of a solid body located in a stream, whereas the momentary stream direction varies along the wave direction. The wave forces are therefore always calculated for a length unit of the structure and summed up to create the base shear force.

Due to the complex geometry of the jacket and the motion of the wave, loads caused by one wave can vary depending on the position of the wave relatively to the jacket. Figure 3-5 shows for example two positions of the same wave.

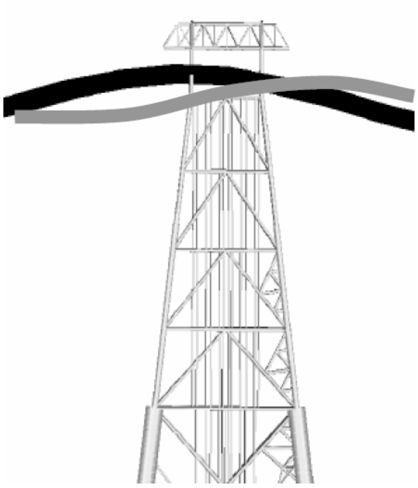


Figure 3-5: Wave stepping, source
"USFOS Hydrodynamic, Theory description of use verification"

The position, at which the maximum base shear is generated, is considered as the representative wave load of this particular wave. This position of wave and the corresponding load is determined by the wave stepping option. Due to the constant height and periodical motion, wave positions can be presented in a polar coordinate system, see Figure 3-6. Wave stepping can thus be defined in terms of discrete phase angle increments. For example, loads shall be calculated for 72 position of a wave, in steps of 5° from 0° to 360°.

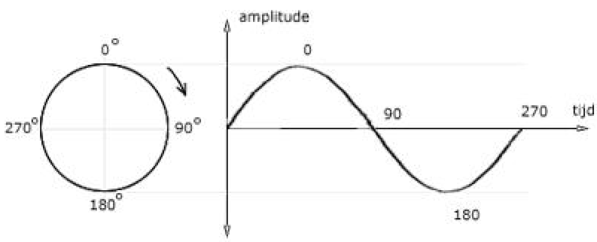


Figure 3-6: Ideal waves with wave height as amplitude

The wave force depends on its height. Hence the environmental design basis is defined in terms of the highest wave being counted on. Typically it is the highest wave within 100 years, or also called the wave with return period of 100 years, based on the long term observation at the site location.

In the calculation procedure, secondary effects are covered by diverse factors which result from empirical and experimental measurements. Factors to be applied are given in ISO 19902 [2]. Especially, the marine growth is taken into account with respect to additional weight, buoyancy, increased diameter of pipe beams and the surface finish.

3.2.2.1 Waves and wave theories

As mentioned above, the wave theories enable to obtain the wave kinematics based on the higher-order mathematical methods. It is traceable to the basic fact of the ocean waves. Waves are caused by disturbances of the fluid. It can be winds or seabed movements, for instance.

> *"Waves are energy in motion. Waves transmit energy by means of cyclic movement through matter. The medium itself does not actually travel in the direction of the energy that is passing through it. The particles in the medium simply oscillate, or cycle, back-and-forth, up-and-down, or around-and-around, transmitting energy from one particle to another."* (Essentials of Oceanography [7], P.239)

Ocean waves are a specific form of waves. They transmit the energy along the interface between two fluids of different density (here atmosphere and water). The particles of ocean waves consist of both longitudinal and transverse components, so that they move in circular orbits in deep water and oval orbits in shallow water. The water particles return to their original position after the time of a wave period. Figure 3-7 displays an ideal wave and the terminology in wave observation and theories.

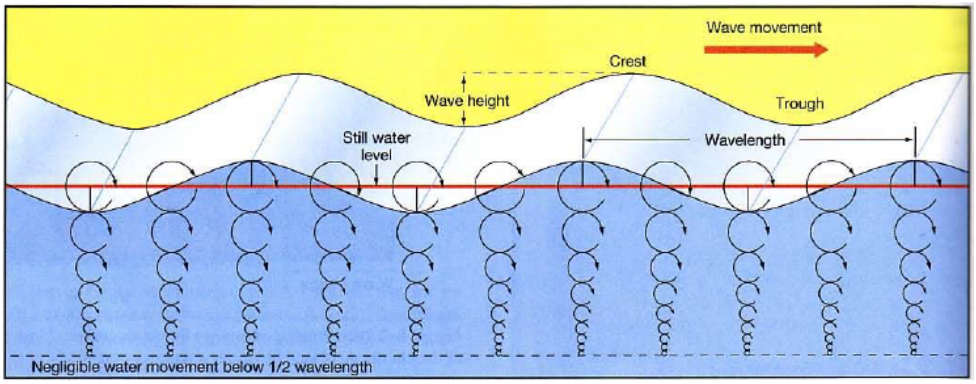


Figure 3-7: Ocean wave motion, source [7] P.240

The length of wave base is a half of the wave length. In deep water, the diameter of the orbits on the surface equals to the wave height and get smaller to negligible when the depth reaches the wave base. Shallow water waves are not given in our case.

The wave impulse induces forces on the jacket, which interrupts the wave movement. These forces are individually calculated for each length unit of the structure and used for calculation of load effects on the jacket. In reality, a sea state is always a superimposition of waves of different directions, height and periods. For limit states analyses, only one wave in one direction is used as one load case.

For each step of a wave, local kinematic is determined with means of wave theories. There are many wave theories recognized based on different mathematical approaches. Each method has a field of application defined by the relation of wave height and wave length. For a standard wave in deep water, Dean's stream functions theory is used to examine nonlinear water waves numerically. It concerns an application of Laplace transformation to solve the stream functions with two nonlinear free surface boundary conditions, constant pressure and wave height constraint, chosen for stationary, two-dimensional, incompressible and eddy-free follows. The order of the stream function is a measurement of how nonlinear the wave is considered. In deep water, the order can be low (3 to 5) while in shallow water it can be greater than 11. This method delivers a plausible mathematical model of waves and therefore reliable model of load for further calculation.

3.2.2.2 The Morison's equation

The jacket is composed of tubular elements. Because the ratio of wave length to member diameter is greater than 5, the wave load can be determined using the Morison's equation. Morison's equation is a formula developed for computing the hydrodynamic load on a cylindrical body. The force refers to a length unit of the pipe and consists of two parts, the drag part and the inertia part. The drag part concerns the friction between the viscose fluid and the surface of the pipe body which is strongly dependent on the velocity of the fluid immediately adjacent to the body. The inertia part concerns the retarding force of the body equal the force exerted by the flow, according to the first Newton's law of motion, which is dependent on the displaced volume of the body and the fluid acceleration immediately adjacent to the body.

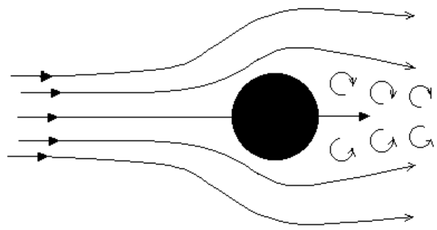


Figure 3-8: Flow over cylindrical body

The velocity of the fluid is to be split into two components, the one is normal and the other is parallel to the member axis. Only the normal component is considered. Figure 3-8 demonstrate the flow component in the direction normal to the member axis. The drag term and the inertia term are harmonised by empirical and experimental coefficients C_d and C_m as followed:

$$F = F_d + F_i = C_d \frac{1}{2} \rho_w U |U| A + C_m \rho_w V \frac{\delta U}{\delta t} \qquad 3.2\text{-}1$$

F	local action vector per unit length acting normal to the axis of the member
F_d	the vector for the drag action per unit length normal to the axis of the member in the plane of the member axis and U
F_i	the vector for the inertia action per unit length normal to the axis of member in the plan of the member axis and $\frac{\delta U}{\delta t}$
C_d	hydrodynamic drag coefficient
ρ_w	mass density of the water
A	effective dimension of the cross-sectional area normal to the member axis per unit length (=D for circular cylinder)
V	displaced volume of the member per unit length
D	effective diameter of a member, including marine growth

U the component of the local water particle velocity vector (due to wave/current) normal to the axis of the member

|U| the modulus (the absolute value) of U

C_m hydrodynamic inertia coefficient

$\frac{\delta U}{\delta t}$ component of the local water particle acceleration vector normal to the axis of the member

Typical values of hydrodynamic coefficients C_d and C_m are given in [2]. The values for smooth surface can be used for member without marine growth, and for rough surface implies member with marine growth, mostly under water line.

Table 3.2-1: Typical values of hydrodynamic coefficients, source [2], Table 9.5-1 (P.55)

Surface of component	C_d	C_m
smooth	0,65	1,6
rough	1,05	1,2

The Morison's forces can be then summed up to determine the base shear of the wave and if needed overturning moment by additional considering the position of the centre of gravity and the moment induced by each single Morison's force.

3.2.3 Accidental Limit States Analysis (ALS)

Besides the usual situations of loads acting on the structure, offshore structures are exposed to various abnormal hazards, such as ship collision, explosion or abnormal environmental actions. Accidental limit state (ALS) among other is a design criterion for steel structure. It aims for a safety function even in the event of accidents along the service life of structure. ALS analysis is also used for evaluating the structure in damage condition, if the damage is tolerable or it requires further checks to investigate the consequence in depth. For example, if a brace become non-structural, it will lead to load redistribution over the structure. ALS analysis is to be carried out for damaged structure exposed to accidental situations. If the structural capacity of other element in after damage situation is overcharged and plastification occurs, a non-linear structural analysis shall be applied. Table 3.2-2 shows the guiding terms for ALS.

Table 3.2-2: Comparison of extreme and abnormal environmental actions, source [2] P.71

Requirement	Situation	
	Extreme environmental actions	Abnormal environmental actions
Governing clause for actions	Clause 9	Clause 10
Limit state	ULS	ALS
Return period	100 years	See 10.1.5, default 10 000 years
Partial action factor	See Clause 9, default 1,35	1,0
Partial resistance factors	See Clauses 13, 14, 15, 17; generally 1,05 to 1,25 but up to 2,0	1,0
Wave crest height	Associated with 100 year return event	Associated with abnormal environmental event

Accidental hazards of different types are categorized into three groups, according to [2], in respect of their probabilities of occurrence or in other words return periods. Although the default return period is 10 000 years, to be chosen in this paper is the event of abnormal environmental actions with return period of 100 years, corresponding to the highest wave within 100 years, in combination with current and wind in the same direction. The procedure of computing loads caused by wave is therefore applicable for both ultimate limit states analysis (ULS) and ALS.

The load factors and resistance factors in ALS are all 1,0 and thus the structure is subjected to actions without a margin of safety. This is a conservative analysis and therefore a reliable method to assess the structural integrity after damage.

3.2.3.1 Redundancy analysis

In the context of structural analyses, a redundancy analysis refers to the member importance analysis. The member importance implies the severity of its absence. It can be proven by the comparison of the intact structure and the structure without the concerned member. The reduction of the structure resistance, as a result, can convey persuasively the consequence of removing the member. It is measured by the reduction of a redundancy ratio as defined in 3.2.3.3.

In the risk assessment, redundancy analysis is applied to the brace involved to the joint with high damage resulting from the FLS. The concerned member is to be removed from structure to produce a damaged structure. It is then subjected to full-capacity analysis and the code check under given conditions of ALS. They include further analyses which help to determine the overall strength of the structure in terms of the lateral actions exerting on the structure. Lateral actions are loads caused by waves, current and wind.

3.2.3.2 Full-capacity analysis

Full-capacity analysis is an approximate method, in which the jacket is subjected to continually increasing lateral forces until a target value is reached, in this case, until the utilization of a member or joint reaches 1,0. This is implemented by applying a load factor, which is sequentially raised in small steps, for the lateral loads in the linear load combination of the ALS. Full-capacity analysis is applied to obtain the force of full capacity (F_{fc}) as defined in the following chapter.

3.2.3.3 Redundancy ratio

In the redundancy analysis, according to [2], the reserve strength check of the structure shall be performed, where reserve strength ratio (RSR) is the measurement of the capacity of a structural system to withstand overloads. The overload here implies the unfactored environmental force which, when co-existing unfactored permanent and variable actions are added, causes collapse of the structure ($F_{collapse}$). To determine $F_{collapse,}$ a progressive collapse limit state analysis shall be carried out. It demands much effort of analytical and comprehensive calculations, where plastification shall be taken into account in non-linear analyses. In this thesis, the principle of reserve strength ratio is applied with a simplified practice.

The aforementioned F_{fc} takes place of the $F_{collapse}$ in this approach. F_{fc} is the unfactored environmental action, at which, when co-existing unfactored permanent and variable actions are added, the unfactored local resistance capacity is utilized. And analogous to RSR, the redundancy ratio (RCR) is defined as the ratio of the comprehensive capacity of the structure to withstand the environmental load with return period of 100 years. RCR is a reserve factor defined as the ratio of the strength to the load. It implies the general safety factor as against worst environmental conditions of the structure.

$$\mathrm{RCR} = \frac{F_{fc}}{F_{100}} \qquad 3.2\text{-}2$$

F_{fc}: is the un-factored global environmental action which, when co-existing un-factored permanent and variable actions are added, where material is utilized to capacity.

F_{100}: is the un-factored 100 year global environmental action calculated in acc. to ISO 19902-clause 9

The F_{fc} is achieved when the utilization of any member or joint reaches 1,0 where the material is working fully to capacity.

RCR supplies an evaluation criterion referring to the local failure, while with RSR it refers to the total failure of the overall system. For the concept of periodic inspection, the initial failure shall be detected and, as soon as possible, assigned for the preventive or corrective measurement to avoid the propagating of cracks. RCR is a transparent valuation for estimating the importance of a member and therefore the consequence of corresponding damage.

3.2.3.4 Code checks

Code check comprises member check and joint check. They are the calculations of member utilization and joint utilization (generalized as component utilization) under given condition of the jacket and the environmental actions. The objective of the code check is to indicate the structural capacity exploitation of each individual tubular member and tubular joint, under particular static (or quasi static) load conditions.

Component utilization is defined, according to [2], as the maximum value of the ratio of the generalized representation of the design stress (force) in a structural component to the generalized representation of the resistance in stress (force) units of the component. Only utilization less than 1,0 satisfies the design criterion for a particular limit state (see formula 3.2-3, source [2] P. 20):

$$U_c = \frac{F_r \cdot \gamma_f}{\frac{R_r}{\gamma_r}} = \frac{F_d}{R_d} \qquad 3.2\text{-}3$$

Uc utilization of component (member or joint)

F_d design stress (force) due to factored actions

R_d design resistance due to divide of representative resistance by resistance factor.

Note In ALS analysis γ_f and γ_r are equal 1,0, F_r is combined force with the environmental load with return period of 100 years.

Tubular members are pipe elements of the jacket, obtained by division of braces or chords into length units. The more finely it is split, the more accurate the check is. Member check is to be carried out according to [2] clauses 13. It contains 9 separate calculations with the formula 3.2-3 applied as the general rule. They differ from each other by the design stresses. Thus the utilization is considered with respect to a single component of stress or to a stress resultant from the combination of them. Table 3.2-3 shows the stress components regarded in the 9 sub-checks of a member check.

Table 3.2-3: Arrangement of requirements for tubular members, source [2] Table 13.1-1

Subclause	Component	Actions				
		Tension	Compression	Bending	Shear	Hydrostatic
13.2.2	Tubular	X				
13.2.3	Tubular		X			
13.2.4	Tubular			X		
13.2.5	Tubular				X	
13.2.6	Tubular					X
13.3.2	Tubular	X		X		
13.3.3	Tubular		X	X		
13.4.2	Tubular	X		X		X
13.4.3	Tubular		X	X		X

The maximum value of these partial utilizations is considered the representative utilization of the investigated member. It is necessary to undertake the member check under various load effects because tubular members of an offshore jacket are seen as slender structure with hollow profile. Along with it, failure modes as column buckling and hoop buckling are adequately secured. The calculation procedure is given in [2] clause 13, included herein as appendix 02.

The joint check is defined quite differently than the member check due to the different failure mode and the complex behaviour of tubular joints. Joint utilization is defined as one totalized utilization in terms of the axial stress component, in-plane and out-plane bending moment summing up, according to [2] clause 14, as follows (source [2], P.148):

$$U_j = \left|\frac{P_B}{P_d}\right| + \left(\frac{M_B}{M_d}\right)^2_{ipb} + \left|\frac{M_B}{M_d}\right|_{opb} \qquad 3.2\text{-}4$$

U_j the joint utilization

P_B axial force in the brace member from actions

M_B bending moment in the brace member from actions

P_d design value of the joint axial strength (see [2] 14.3.2); equal P_{uj} for ALS

P_{uj} the representative joint axial strength, in force units

M_d design value of the joint bending moment strength (see [2] 14.3.2); equal M_{uj} for ALS

M_{uj} the representative joint bending moment strength, in moment units

ipb represents in-plane bending moment and strength

opb represents out-plane bending moment and strength

This check aims to avoid the overloading of the material shear strength due to forces in the though thickness direction, also called punching shear. Thus the force exerting on the joint is calculated only in terms of the axial force in the brace of the joint. The force in the chord of the joint is taken into account in form of a chord factor applied for calculating the joint representative strength.

While the member strength is dependent on the kind of stress component, except the Euler’s buckling, the joint strength is strongly parameterized by the geometry and the force flow through it.

> *“The strength of a joint varies not only with its materials and geometry but also with the pattern of forces on each brace. Consequently, these strengths can vary between load cases.”* (ISO 19902 [2], P.144)

Other than joints considered in the FLS, a joint under joint check can consist of more braces. For multiplanar joints, each plane is observed individually. The adjacent planes do not have influences on the each other. To prevent the punching through, the chord is usually reinforced at the area of the joint with a larger wall thickness or heavier cross section called the chord can. It is to be considered for Y- and X- joints. Figure 3-9 shows the terminology and geometrical parameters of a simple circular tubular joint in the utilization calculation. Figure 3-10 shows the three basic joint types classified by the force flow.

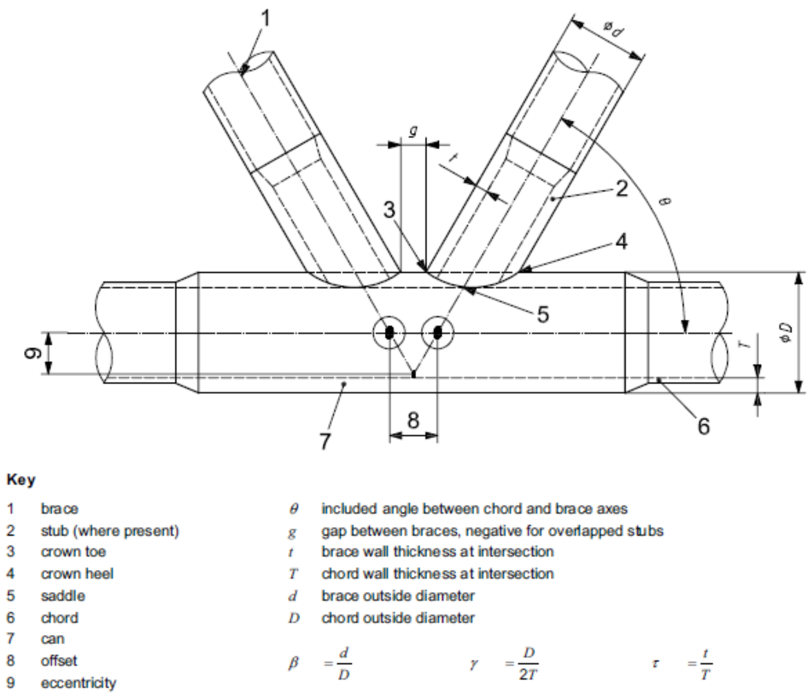


Figure 3-9: Terminology and geometrical parameters for simple tubular joints, source [2] figure 14.1-1

Simple tubular joints are joints having no gussets, diaphragms, grout or stiffeners. Simple Y- and Y- joints have no overlap of principle braces, but simple K-joints may have overlap up to 0,6 D ([2], P.143). A joint can be both a Y– or K-joint. It depends on whether the load at brace is taken in the chord or in the other brace or in both.

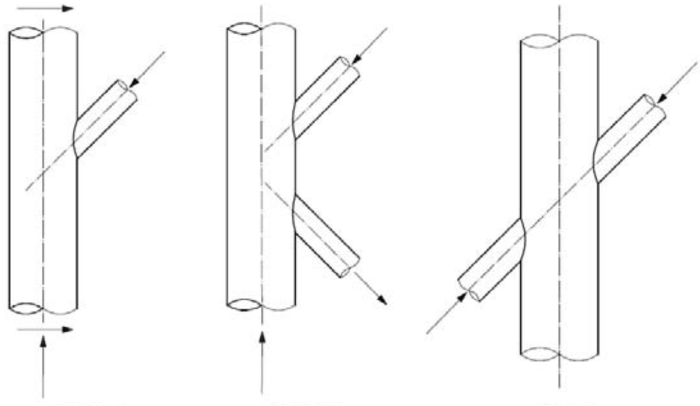


Figure 3-10: Basic planar joint types, source [2] figure 14.2-1

As mentioned above, the joint strength is determined in terms of the material properties, the geometrical parameters and the load pattern in the joint. Joint strengths against the axial brace force and against the bending moment are to be calculated according to Equation 3.2-5 and 3.2-6, ([2], P.144). In the ALS, where partial resistance factors are 1,0, the design values of joint strength (P_d and M_d as in Equation 3.2-4) are equal the values of representative joint strengths as follows:

$$P_{uj} = \frac{f_y . T^2}{sin\theta} Q_u . Q_f \qquad 3.2\text{-}5$$

$$M_{uj} = \frac{f_y . T^2 . d}{sin\theta} Q_u . Q_f \qquad 3.2\text{-}6$$

P_{uj}	representative joint axial strength, in force units; P_d in ALS
M_{uj}	representative joint bending moment strength, in moment units; M_d in ALS
f_y	representative yield strength of the chord member at the joint
T	chord wall thickness at the intersection with the brace
d	brace outer diameter
θ	included angle between brace and chord
Q_u	strength factor
Q_f	chord force factor

The determination of the strength factor (Q_u) and the chord force factor (Q_f) is given in [2] sub-clauses 14.3.3 and 14.3.4.

The effect of a chord can on the strength of a Y- or X joint is additionally considered through a term, which is again dependent on the geometrical parameters of the joint and the chord can. Especially herein, an effective chord length is to be calculated because the force flows through to the chord. A part of the chord, which is not the real length of the chord can, would have to carry this through flowing load. Detail formulation is given in [2] sub-clause 14.3.5. The complete calculation procedure is given in [2] Clause 14, included herein as appendix 04.

3.2.4 Fatigue Limit States Analysis (FLS)

3.2.4.1 General

Until now it has merely been dealt with the kind of structural strength that is against a provided constant stress caused by a provided constant load. The structure thus is considered intact as long as the force does not exceed the respective strength of the structure. But even if the stress is smaller than the allowable stress, the repeated variation of it can lead to material failure. Concerning varying loads, the structure feature another kind of strength- the fatigue strength.

> *"Fatigue of material means a process characterized by a gradual reduction in the capacity of the material to withstand repeated loads. Damage means the reduction in strength after a certain number of repeated loadings."* (Offshore Structures vol. 2 [13], P.211)

The variation of existing stress evokes in materials the sliding of micro textures. If this lasts for a period of time, micro fracture may initiate in material as well as on the surface of the structure. Once the fracture occurs, it can propagate rapidly and come to visual cracks. This phenomenon is explicitly observed in experiments and studies of fatigue. Fatigue strength or fatigue limit refers to the number of cycles of stress that a structural specimen can withstand before the failure occurs. In the reality, structures are usually exposed to various different stress ranges with different numbers of loading cycles. Under given circumstances, the cumulative damage is used to derive the fatigue resistance and the fatigue life of the structure.

There are many methods of fatigue assessment such as method using S-N data, using fracture mechanism. In this thesis, the method using S-N curve data according to [2] clause 16 is applied. The fatigue assessment with this method supplies a transparent and reliable prediction of the structure condition for its service life and therefore a clear identification of potential failure for the risk assessment.

Fatigue limit analysis (FLS) is the analysis in which the structure is subjected to varying loads in terms of the number of cycles and the loading ranges. Load effects in the form of stress ranges are then computed for particular locations on the structure. With means of the S-N curve data and the stress ranges as well as corresponding numbers of load cycles, fatigue damage at the investigated location can be estimated. Figure 3-11 shows varying stress in the allowable margin of the tensile strength.

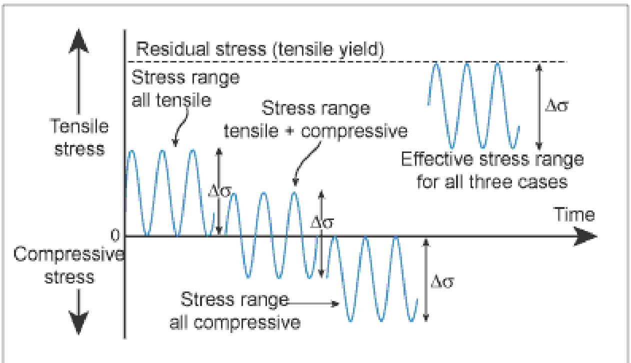


Figure 3-11: Stress ranges, source www.twi-global.com at 13:08 on 31.07.2015

The stress range is defined as the difference between the maximum and the minimum values of stress induced by a loading case. Stresses induced by permanent loads are therefore omitted in the FLS. They shall be smaller than the allowable stress and so do the maximum peak stress. This is to be ensured by the design. One of the design criteria for offshore structure is the ultimate limit. The structure shall be able to withstand the combined load which may occur within 100 years. The stress ranges inducing fatigue are more often periodical loads and always lie within the stress limits. Waves are the major and dominant causes of varying loads. Other factors such as vortex induced vibrations or wind induced vibrations are not considered herein.

A wave has a constant mean height and a mean returning period. Hence the load effects on the structure they cause have the mean stress ranges, which are constant at each location on the structure. In the chapters afore, it is noted that weld seams especially at joints sensitive against fatigue. The inhomogeneity in material may fortify the sliding of material texture and the notches on the surface and also in the material may cause the increased stress ranges locally. Besides these facts, a tubular joint features the redirection of force flow in the material and can lead to stress concentration at the turning point. Due to it, the tubular joints are always more fatigue critical. The FLS is thus undertaken solely for tubular joints of the offshore jacket.

The jacket bears in the seabed by means of the piles burrowed into the soil. The reaction forces of the bearing behave in line with the damping property of the seabed. The structural response of the installation deviates therefore from the response of which with fixed bearing. For plausible results in calculating stress ranges, the behaviour of the structure shall be as modelled as accurately as possible. It is mostly realized by the defining boundary conditions based on the knowledge about soil rigidity and its impact on the structure.

Following sub-chapters describe in more detail the methods used.

3.2.4.2 S-N curve

In this thesis, the assessment fatigue strengths is done with the method using S-N curve data according to [2]. Normally in a static analysis, safety factors are included in of the calculation, as the safety factor applied for allowable stress or as the load and resistance factors. In a FLS, the safety factor is already included in to design curve of the S-N curve, see Figure 3-1, Chapter 3.1.2. The representative resistance of the structure against fatigue does not relate to the yield strength of the material but to the ability to endure the exposure stress ranges, in terms of the number of stress cycles. This is also included in the design S-N curve.

> *"Safety against fatigue failure is provided by using fatigue resistance in the form of a design S-N curve. Overall safety against failures associated with fatigue damage accumulation is further provided by an additional fatigue design factor larger than1,0. Local experience can be taken into account by a local experience factor k, which can be larger or smaller than 1,0."* (ISO 19902 [2], P.169)

Because it does not concern a design calculation herein but a fatigue check of existing jacket, there is no design factor considered. The jacket is subjected to a FLS under provided conditions of waves. As the results, the damages of joints are calculated without consideration of fatigue design factors for both inspectable as well as not-inspectable joints. The fatigue check gives just a tendency for the inspection of accessible parts of the jacket. Not accessible parts shall have been adequately dimensioned at the stage of design.

Figure 3-12 displays a general S-N curve in a double logarithmic plot. A S-N curve is a record of experiments with a structural specimen exposed to a varying load, which generates a constant stress range. In this process the load cycles are counted until a fracture appears, called the number of cycles to failure. It is the fatigue strength of the specimen against this stress range. The experiments of various loads are further conducted to complete the curve. The relationship between a constant stress rang (S) and the corresponding fatigue strength (number to failure – N) is gathered and represented in the S-N curve for this particular specimen. Corrosion and notch effects are assumed as not existing.

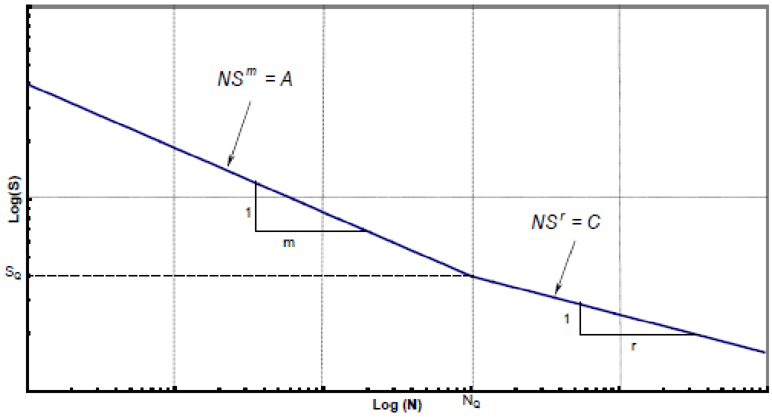


Figure 3-12: S-N curve, source [8] P.7

According to [2], S-N data is presented in form of an equation, in which the fatigue strength is a mathematical function of the exposure stress range, see Equation 3.2-7. The parameters k_1 and m vary in respect of specimen types. Table 3.2-4 provides the values of these parameters for tubular joints employed in air as well as under water.

$$\log_{10} N = \log_{10} k_1 - m.\log_{10} S \qquad 3.2\text{-}7$$

Table 3.2-4: S-N curve paramerters for tubular joints, source [2] P.186

Curve	Air		Sea water with adequate corrosion protection	
	$\log_{10}k_1$ (for S in units of MPa)	m	$\log_{10}k_1$ (for S in units of MPa)	m
Tubular joints (TJ)	12,48 16,13	3,0 for $N \leqslant 10^7$ 5,0 for $N > 10^7$	12,18 16,13	3,0 for $N \leqslant 1{,}8 \times 10^6$ 5,0 for $N > 1{,}8 \times 10^6$

Normally steel material features an endurance strength, which implies the endless intactness of material if it is exposed to a stress range under a particular value. But steels employed in offshore conditions, especially steel welded seam, do not have this nature.

It is due to the uncertainties in material of welds and the corrosive environment. The S-N design curve used in [2] is explicitly made for tubular joints with outer fillet welds.

Fatigue damage is defined as the ratio of the number of cycles experienced to the number of cycles to failure. This damage refers to the degree of impairment, which is caused by solely the concerned stress ranges. Offshore installations are however exposed to various waves having different periods and inducing different stress ranges. The cumulative damage is therefore used as the representative damage which comprises damages caused by single stress ranges. This is further explained in the following Chapter 3.2.4.3 - Cumulative damage.

3.2.4.3 Cumulative damage according to Palmgren-Miner

As stated earlier, a structure is always exposed to different stress ranges which repeat in different rhythms. After a certain period of time, it suffers a number of damages which result from all of exposure load cases. These damages contribute to a total damage, also called cumulative damage. The cumulative damage of an individual area of the structure is calculated based on the Palmgren-Miner rule:

$$\mathrm{D} = k_{LE}.\gamma_{FD}.\sum_i \frac{n_i}{N_i} \qquad 3.2\text{-}8$$

D	damage ratio for a period T
k_{LE}	local experience factor, default is 1,0
γ_{FD}	fatigue damage design factor, see table 3.2-2
n_i	number of cycles of stress range $S_{i,}$, occurring during time T
N_i	number of cycles to failure under constant amplitude stress range S_i taken from the relevant S-N curve.

The cumulative damage of the structure at a certain point is the ratio of the number of cycles in a period T at a given stress range to the number of cycles to failure at that stress range, summed up over various stress ranges experienced during the period T. With consideration of the fatigue design factor and the local experience factor, the accumulated damage shall be less than or equal to 1,0 to avoid the rapidly propagating cracks.

The application of the S-N curve requires high accuracy of determining stress ranges. Due to the angular design, the actual stress at the corner area is elevated. It refers to the largest value of stress at the intersection between brace and chord of the joint, which decides on the fatigue strength at this spot and consequentially on the fatigue strength of the joint. Hence the actual local stress at tubular joints is specifically scrutinized.

3.2.4.4 Stress concentration factors (SCF) of welded tubular joints according to Efthymiou's equations

This sub-chapter deals with stress concentration in the context of fatigue assessment for welded tubular joints. The stress peak on the upper layer of the joint is to be validated to be then used in the S-N curve. The weld root is assumed to have no void and failure and to have full penetration.

Stress concentration is the elevated density of force flow in material caused by geometric discontinuities. The force flow is unequally distributed over the cross section and raises local agglomeration of internal forces in adjacent to these areas, and as a result, the local increased stress. Figure 3-13 shows the force flows in material without and with discontinuity in general.

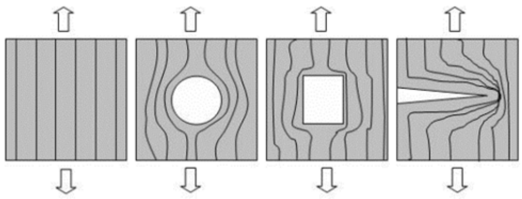


Figure 3-13: Force flow with geometrical uncertainty, source www.enventure.com on 12.08.2015

For tubular connection, the geometric discontinuity is the redirection of the force pattern and variation of the cross section. The locally increased stress can be determined by multiplying the nominal stress in the component with a concentration factor. This stress concentration factor (SCF) is therefore defined by the ratio of the maximum stress to the nominal stress in the component. It is to bear in mind, that the SCF does not only depend on the joint geometry and joint type, but also the type of stress components of the brace (axial, in-plane bending, and out-plane bending). It also depends on the application for seams at the chord side or brace side, and on the specific location.

$$\mathbf{SCF} = \frac{\text{the range of the GS at a particular location of the intersection weld (excluding notch effect)}}{\text{the range of the nominal brace stress}}$$

The local increased stress caused by the joint geometry (geometric stress - GS), which is to be used for S-N curve, can be therefore determined by calculating the nominal stress components and the corresponding SCFs, in accordance with the structural hot spot stress concept (geometric stress range concept – GSR concept). The geometric stress resultant is then determined by the combination of the stress components multiplied with their corresponding SCFs.

> *"The GSR concept has evolved as the most practical basis for fatigue design of tubular joints. This concept places many different structural geometries on a common basis, enabling them to be treated using a single S–N curve. The basis of this concept is to capture a stress (or strain) in the proximity of the weld toes, which characterizes the fatigue life of the joint, but excludes the very local microscopic effects such as the sharp notch, undercut and crack-like defects at the weld toe. These local weld notch effects are included in the S–N curve."* (ISO 19902 [2], P.448)

In the "Recommendation for fatigue design of welded joints and components" [5], the International Institute of Welding (IIW) also recommends to use the structural hot-spot stress concept for determining SCFs for tubular joints ([5], P.24). This concept is developed to investigate the increased stress at the surface in the vicinity of the weld seam, based on the linear extrapolation of stress distribution in the beam outside of weld seams. Figure 3-17 displays the terminology and stress distribution in the brace and in the chord of a tubular joint. Here it can be seen, that instead of considering the exact weld geometry the hot spot stress (or also geometric stress) is determined in terms of the nominal stress in brace or in chord and the constant concentration factor SCF.

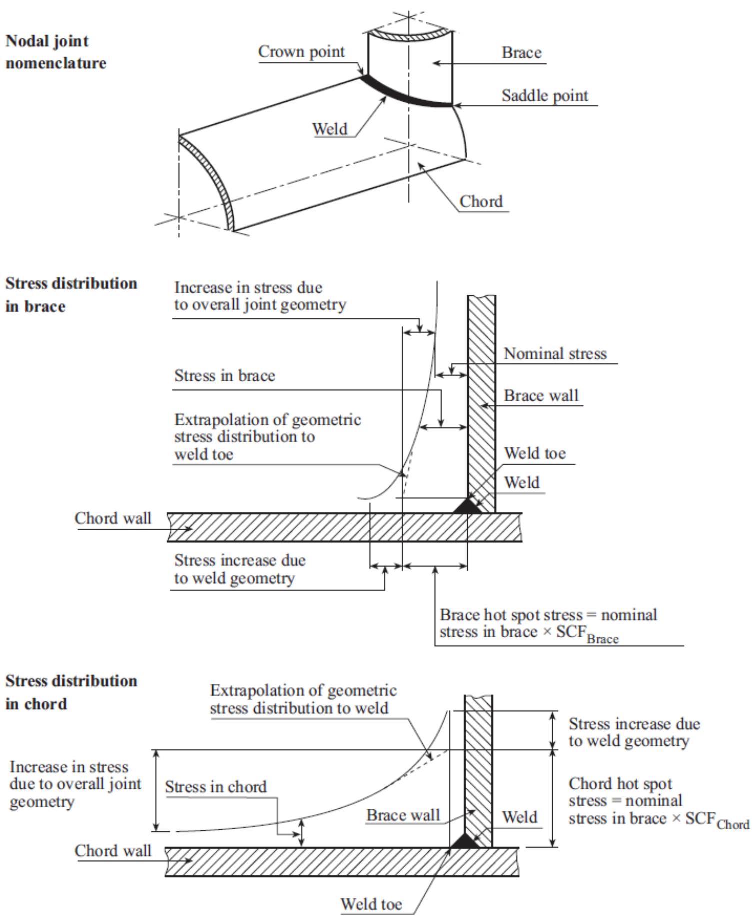


Figure 3-14: Structural hot-spot stress concept, source GL-IV-7

The advantage of this method is that the stress growth can be determined for complex joints without giving a detailed notch class. Therefore it is applicable for structures with a huge number of joints.

> *"It is assumed and confirmed by experiments that weld of a similar shapes have the same fatigue behaviour so that a single design S-N curve can be employed for any weld class…. There is no need to directly consider the stress concentration effects of the weld."* (Fatigue of Welds [11], P.6)

SCFs may be derived from finite element analyses (FEA), model tests or empirical equations based on such methods ([2], P449). FEA require either a solid-element model or a shell-element model. These models demand a high computing capacity and particularly the geometry of weld, which is actually not always provided. They are usually used for investigating one particular joint and are not convenient for an entire jacket. Offshore jackets are modelled with beam-elements. They require another method.

Efthymiou's equations are parametric formulae based on the structural hot spot concept and experiments. As per Efthymiou's equations, the SCFs are defined in terms of geometrical parameters of the joint as shown in Figure 3-15.

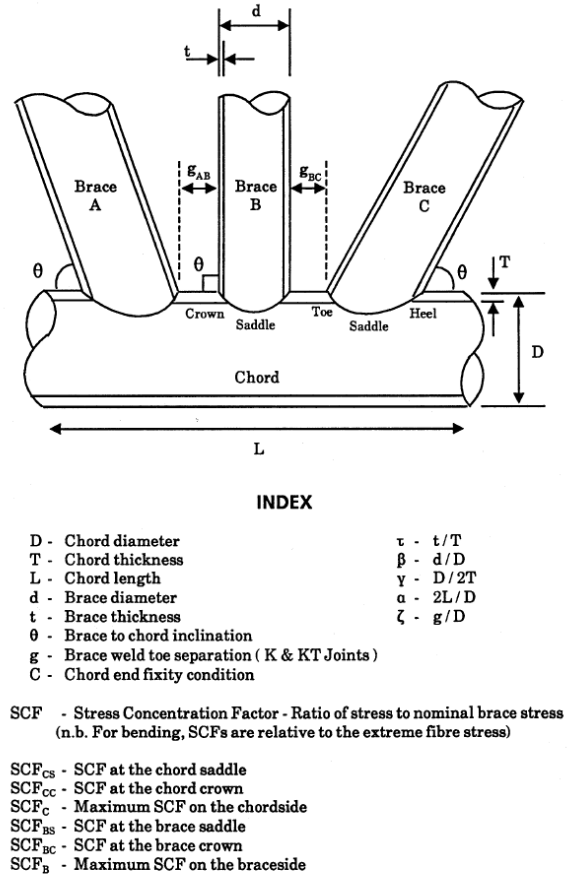


Figure 3-15: Geometrical parameters of joints, source [11]

The Efthymious' equations are however only applicable if these geometrical values fall within a validation range. For K joints, they are:

$$0.2 \leq \beta \leq 1.0$$
$$8 \leq \gamma \leq 32$$
$$0.2 \leq \tau \leq 1.0$$
$$20° \leq \Theta \leq 90°$$
$$4 \leq \alpha \leq 40$$
$$0.0 \leq \zeta \leq 1.0$$

The Efthymious' equations provide formulae for calculation of 16 SCFs, see ble 3.2-5. For joints with several braces, the forces from one brace are taken into the other one, either completely or partially (this is termed as balanced or unbalanced loads). SCFs are to be calculated accordingly. For calculating the geometric stress at a circumferential point of weld seam around the brace, only 4 of 16 SCFs are selected and applied. The questions are if the loads in braces are balanced and if the investigated point is on the chord side or the brace side of the weld. The calculation procedure is given in [2] clause A.16.10.2.2.2, included herein as appendix 04.

Table 3.2-5: SCFs for individual components of stress for each type of joint

side	axial		in-plane bending	out-plane bending
	at crown points	at saddle points	around saddle points	around crown points
				balanced loads
chord	SCF1	SCF2	SCF3	SCF4
brace	SCF1b	SCF2b	SCF3b	SCF4b
				unbalanced loads
chord	SCF1u	SCF2u	SCF3u	SCF4u
brace	SCF1ub	SCF2ub	SCF3ub	SCF4ub

To determine the geometric stresses on both sides of the seams, the SCFs are used to magnify the nominal stress in brace in each corresponding component. The nominal axial stress in chord is covered at the crown point of the brace end.

> *"The effect of the nominal chord member stresses is adequately covered by superimposing the axial chord member stress (in the chord can) to the chord crown location only, following a consistent sign convention, i.e. tensile contributions are positive. Other influences, namely at the saddle location or the brace side locations and the contribution from nominal bending stresses in the chord can, are either considerably smaller or are already accounted for because they arise from brace loading."* (M. Efthymiou [9], P. 2-7 & 2-8)

The nominal stress in brace is provided by the global analysis and to be combined with the SCFs according to joint and loading type as in Equation 3.2-9. Figure 3-16 pictures the tubular seam around the brace end with numbered positions. The desired position for geometric stress calculation is named with "i". This specification of position can be advantageous for localization of critical positions, so that grinding work can be done.

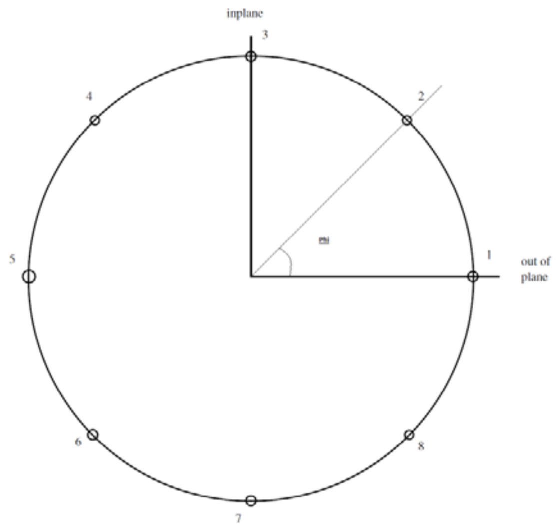


Figure 3-16: Positions on a weld seam for both sides, with 1&5 are saddle points, 3&7 are crown points

The geometrical stress at the circumferential points indicated in Figure 3-16 are superposition stresses derived by summation of the stress components combined with the associated SCFs. The Equations 3.2-9 are used for calculation of hot spot stresses at 8 points around the tubular weld seam on the chord side under balanced loads. For other particular side and case of loads, the SCFs from Table 3.2-5 are to be selected accordingly.

$$\begin{aligned}
\sigma_1 &= \sigma_a.\ SCF_1 + \sigma_{ipb}.\ SCF_3 \\
\sigma_2 &= \sigma_a.\ \frac{1}{2}.(SCF_1 + SCF_2) + \sigma_{ipb}.\ \frac{1}{2}.\sqrt{2}\ SCF_3 - \sigma_{opb}.\ \frac{1}{2}.\sqrt{2}\ SCF_4 \\
\sigma_3 &= \sigma_a.\ SCF_2 - \sigma_{opb}.\ SCF_4 \\
\sigma_4 &= \sigma_a.\ \frac{1}{2}.(SCF_1 + SCF_2) - \sigma_{ipb}.\ \frac{1}{2}.\sqrt{2}\ SCF_3 - \sigma_{opb}.\ \frac{1}{2}.\sqrt{2}\ SCF_4 \\
\sigma_5 &= \sigma_a.\ SCF_1 - \sigma_{ipb}.\ SCF_3 \\
\sigma_6 &= \sigma_a.\ \frac{1}{2}.(SCF_1 + SCF_2) - \sigma_{ipb}.\ \frac{1}{2}.\sqrt{2} SCF_3 + \sigma_{opb}.\ \frac{1}{2}.\sqrt{2}\ SCF_4 \\
\sigma_7 &= \sigma_a.\ SCF_2 + \sigma_{opb}.\ SCF_4 \\
\sigma_8 &= \sigma_a.\ \frac{1}{2}.(SCF_1 + SCF_2) + \sigma_{ipb}.\ \frac{1}{2}.\sqrt{2}\ SCF_3 + \sigma_{opb}.\ \frac{1}{2}.\sqrt{2}\ SCF_4
\end{aligned} \qquad 3.2\text{-}9$$

σ_i	geometric stress at the point i (1 to 8)
σ_a	nominal axial stress in brace
σ_{opb}	nominal out-plane bending stress in brace
σ_{ipb}	nominal in-plane bending stress in brace

The nominal stresses in brace can be taken from the global analysis. Being subjected to a cycle of wave load, maximum and minimum of the geometric stresses can be determined and, as the result, the geometric stress ranges for particular locations can be determined to be incorporated consistently into the S-N curve.

The 16 SCFs can be derived from the geometrical parameters of the jacket, while the global analysis supplies nominal stresses and load patterns needed for the computation. The global analysis in FLS is a number of quasi static analyses, whereby the load cases are waves occurring in the period T and the numbers of load cycles are counted statistically over the period T. The wave data are usually collected and reflected in a scatter diagram.

3.2.4.5 Scatter Diagram

Scatter diagram is a table listing the occurrence of sea-states in terms of significant wave height and wave peak period or mean crossing period (API 2RD, 2.edt).

Firstly the real sea states are irregular waves calling for a representative height and period, so that they become compatible for the quasi static analysis as presented in Chapter 3.2.2. Figure 3-17 shows an example of irregular wave and the derived significant height (H_s), as the average of 1/3 of the greatest height of a wave, and mean period up-crossing the mean sea level (T_z).

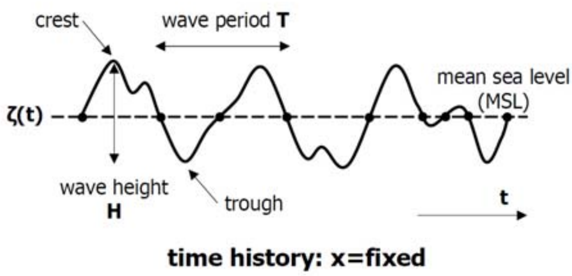


Figure 3-17: Irregular waves, source Internet

Scatter diagram is a presentation mode of waves that occurred in an observation period.

> *"Wave scatter diagram usually represent the long-term wave environment during a (typical) year and should be based on several years of site-specific data in order to ensure that they adequately represent the wave environment at the location of the structure. The data can be determined by measurements, the hindcast modelling or by a combination of the two."* (ISO 19902 [2], P.171)

Figure 3-18 displays an example of scatter diagrams used in FLS of offshore structures, in which the duration of each wave with associated Hs and Tz are provided on a percentage basis, so that they can be flexibly applied for any period of T. The directions of waves are also given,

		0	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	Total
		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	
8.5	9.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
8.0	8.5	0	0	0	0	0	0	0	0	0	0	0	0	0.002	0	0	0	0	0	0	0	0.002
7.5	8.0	0	0	0	0	0	0	0	0	0	0	0.002	0.006	0.004	0	0	0	0	0	0	0	0.012
7.0	7.5	0	0	0	0	0	0	0	0	0	0	0.004	0.006	0.006	0	0	0	0	0	0	0	0.016
6.5	7.0	0	0	0	0	0	0	0	0	0	0.002	0.010	0.012	0.004	0	0	0	0	0	0	0	0.028
6.0	6.5	0	0	0	0	0	0	0	0	0	0.008	0.034	0.026	0.004	0	0	0	0	0	0	0	0.073
5.5	6.0	0	0	0	0	0	0	0	0	0	0.040	0.111	0.026	0.010	0	0	0	0	0	0	0	0.187
5.0	5.5	0	0	0	0	0	0	0	0	0.012	0.187	0.131	0.020	0.004	0	0	0	0	0	0	0	0.354
4.5	5.0	0	0	0	0	0	0	0	0.002	0.141	0.393	0.123	0.026	0.006	0	0	0	0	0	0	0	0.691
4.0	4.5	0	0	0	0	0	0	0	0.030	0.630	0.367	0.091	0.030	0.004	0.004	0	0	0	0	0	0	1.156
3.5	4.0	0	0	0	0	0	0	0.004	0.322	1.186	0.316	0.089	0.022	0.008	0.006	0.004	0	0	0	0	0	1.957
3.0	3.5	0	0	0	0	0	0	0.103	1.770	1.156	0.296	0.073	0.026	0.002	0.010	0.016	0.006	0	0	0	0	3.457
2.5	3.0	0	0	0	0	0	0.030	1.025	3.232	1.023	0.316	0.089	0.018	0.004	0.026	0.010	0	0	0	0	0	5.773
2.0	2.5	0	0	0	0	0.008	0.860	3.908	2.805	1.045	0.332	0.121	0.060	0.042	0.048	0.018	0.006	0.002	0	0	0	9.256
1.5	2.0	0	0	0	0.002	0.803	4.553	4.684	2.632	1.077	0.322	0.173	0.117	0.050	0.026	0.002	0	0	0	0	0	14.441
1.0	1.5	0	0	0	0.628	6.441	4.738	4.555	2.475	1.192	0.582	0.296	0.171	0.044	0.032	0.016	0.004	0.004	0.004	0.004	0	21.186
0.5	1.0	0	0.006	0.731	8.032	4.627	5.358	3.949	1.617	1.027	0.735	0.439	0.149	0.064	0.127	0.103	0.097	0.069	0.044	0.012	0	27.185
0.0	0.5	0	0.548	3.461	2.984	2.265	1.416	0.785	0.646	0.431	0.469	0.211	0.159	0.191	0.240	0.205	0.103	0.062	0.032	0.016	0	14.226
Total		0	0.554	4.192	11.646	14.145	16.954	19.012	15.530	8.920	4.365	1.995	0.876	0.451	0.520	0.375	0.215	0.137	0.081	0.032	0	100

Table 16 Percentage of occurrence of conditions in significant wave height interval, Hs (rows) and peak period interval, Tp (columns). Lower & upper bounds of intervals in first two columns/rows respectively. Season : All. Number of samples: 49664. Colours are intended to enable visualisation of the highest probabilities (red) and lowest probabilities (blue).

Figure 3-18: Scatter-diagram for wave periods actually used, source "BMT AGROSS; Metocean and environmental conditions for Dolwin West (North Sea); RP_A10000_NS_Dolwin_West

By means of the scatter diagram, the number of load cycles can be determined to be incorporated into the S-N curve to the induced stress ranges. Figure 3-19 summarizes the work steps of calculating damages of joints as described. This procedure is applied for one wave with associated hydrodynamic load and number of cycles derived from the scatter diagram. Hence the damage obtained is the damage due to this wave. This process is run for all other load cases to gain the single damages, which contribute to the cumulative damage at the end.

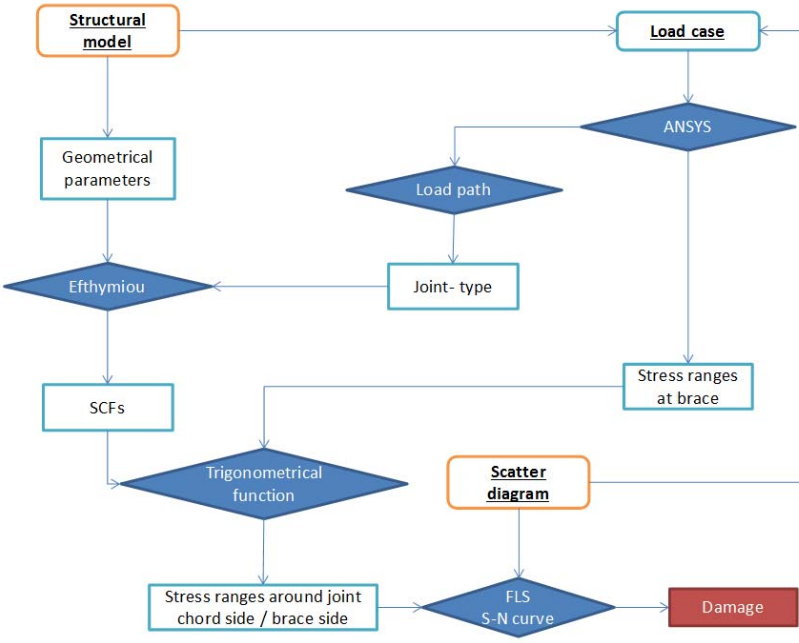


Figure 3-19: Calculation of damage at joints due to one wave

3.3 Calculation with FEM

3.3.1 The FE-model

The employed jacket model is developed with finite element method (FEM) by the Department Offshore Installations of the company DNV GL based on the satisfaction of design criteria for the location North Sea in the German exclusive economic zone and the Metocean Report for the North Sea. The model is constructed mostly with beam-elements, which is appropriate for slender structures, with the exception of shear plates and mud mats featured as shell-elements. The cross section has a tubular hollow profile and generally a constant area. Both beam- and shell-elements offer 6 degrees of freedom, and allow translation as well as rotation at each end. This enables to calculate the components of stress resultants separately, including in-plane and out-plan bending stress of the elements.

Advantages of using beam-elements are time-saving modelling and little effort for calculation. However a jacket model with beam-elements generates in-chord materials at the connections of beams. The end of a brace element is produced up to the axis of the chord and the brace material is therefore partially penetrated through the chord wall and it creates an unrealistic part of the brace. This part of the brace cannot be subtracted from the model due to the connecting function, but it can be defined having no density. Also the connections are the subject to stress concentration, which require parametric equations. This limitation is however removed by high plausibility of parametric formulae. There are many studies which compare the results of this method to those of FEA, and they fit together very well.

The coordinate system is to be orientated, so that the y-axis (green) points to the North and x-axis (red) points to the East. The directions of waves according to scatter diagram shall be adjusted while defining load cases. The load directions are to be derived from the compass direction into angles relative to the x-axis. For ALS, the load direction can be arbitrarily chosen, in this thesis, in 0°, 45° and 292,5°. They are waves, wind and current loads simultaneously exert on the jacket, which come from the West to the East, from the South West to the North East and from the North North West to the South South East. Detail modelling information is given in following sub-chapters.

3.3.1.1 The structural modelling

The jacket itself consists of 4 main legs trapezoidally standing in the water and connected to each other by horizontal and diagonal struts. They are designed as tubular bars and reinforced at the areas of joints with greater wall thickness when they serve as chords. This is visualized in Figure 3-20. The jacket is fixed to the piles, which are driven into the seabed, through 4 shear plates, mud mats and the guiding tubes.

There are also cable guiding tubes called J-tubes attached to the jacket, which do not carry loads. The J-tubes are used for collecting electric cables from the windfarm. They contribute to weight effects on the jacket and as disturbances of the water flow and this is in return has an impact on the wave kinematic locally at the areas of the tubes.

Dimensions of the jacket components are given in Table 3.3-1. The material S355 is selected for tubes acc. to DIN EN 10255 with properties stated in Table 3.3-2.

Table 3.3-1: Dimension of tubular elements

Group	Element	Size OD [mm] x WT [mm]
Primary elements	legs	1650 x 25 Reinforcement at tube joints above (can) LAT: 1650 x 35 Reinforcement at tube joints under (can) LAT: 1650 x 45
	horizontal braces	1100 x 20 Reinforcement at the tube joints: 1100 x 30
	horizontal diagonal braces	900 x 20
	vertikal diagonal braces	1000 x 20
Secondary elements	cable guide tubes	600 x 12
Foundation elements	piles guide tubes	2650 x 75
	piles	2350 x 85

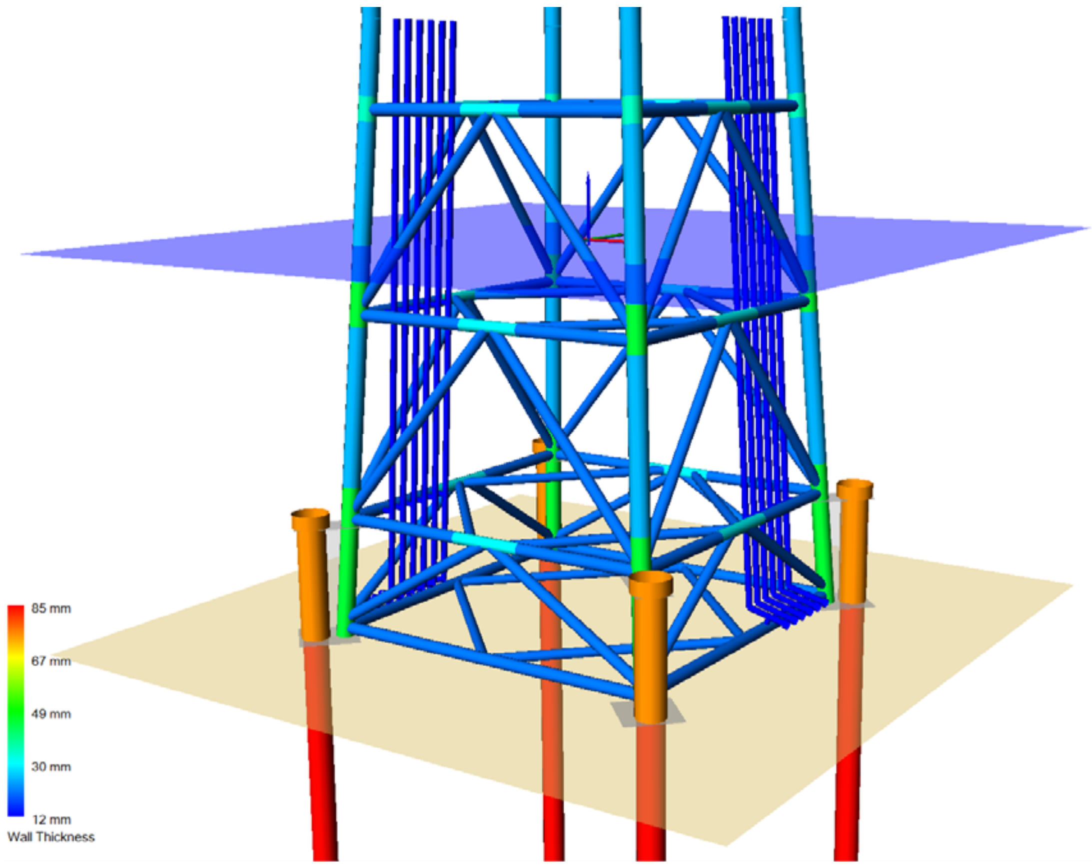


Figure 3-20: Wall thickness

Table 3.3-2: Material properties of S355

Parameters		Quality
Young's modulus		210Gpa
Poisson's ratio		0.3
Density		7850kg/m³
Yield strength	t ≤ 25mm	355Mpa
	t > 25mm	345Mpa
	Chord bottom & Reinforcement	410Mpa
Tensile strength	general	450Mpa
	Chord bottom & Reinforcement	520Mpa

3.3.1.2 The boundary conditions

For fixed offshore installations, piled foundation is usually used. There are two main types of piled foundations: displacement piles and non-displacement piles. They differ from each other by the action towards the soil, either pressed aside or removed to leave the space for piles. Otherwise, the behaviour of single piles and pile groups are to be distinguished. They behave differently and require different approaches for calculating the bearing capacity. Despite it, the bearing capacity depends strongly on the soil properties which vary along the depth under the ground and are represented by various coefficients. Methods and calculations are given by the geotechnical engineering and are not included in the scope of this thesis. Hence this chapter states only roughly the method of simulating the soil-piles interaction in the structural analyses with FEM.

In our case, piles are assumed to act as single piles and to be driven into sand. Generally, piles are affixed to the seabed by the surface friction against the vertical load upwards and against the vertical load downwards by the soil rigidity at pile annuluses. Actually, a check of pile-punch-through shall be done to make sure that the piles are adequately supported in vertical direction. It is herein assumed to be given.

As already mentioned, the reaction forces of the piles foundation interact in accordance with the soil characteristics. Because of the misalignment and flexibility of sand, rocks or clay especially in water, soil-piles interaction shall be considered in the modelling of the supporting by piles foundation.

> *"There are two principle approaches to the computer-based modelling of isolated piles under lateral loading. These are (1) idealization of the soil resistance as discrete springs, and (2) consideration of the soil as an elastic continuum."* (Foundation Design and Construction [14], P.318)

Considering soil as an elastic continuum requires however the modelling of soil medium around piles and high capacity of computation. This method is effortful and thus not appropriate for structural analyses. Due to damping characteristics, the practicing of soil on piles is ideally modelled as spring bearing in horizontal direction. Figure 3-21 shows the p-y curve, which describes the relationship between p-the soil resistance and y-the lateral deflexion (b). On the left side (a) the supporting of piles is modelled as discrete springs representing soil resistance. The spring parameters depend on the soil properties of each layer. It will not be clarified further in this thesis.

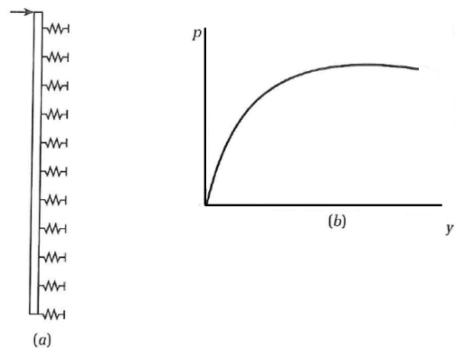


Figure 3-21: Spring model for pile carrying lateral loading, source [14] P.318

Figure 3-22 shows the jacket as a whole including the mean water level and the layer of the seabed. The springs are also outlined representing the spring bearing model.

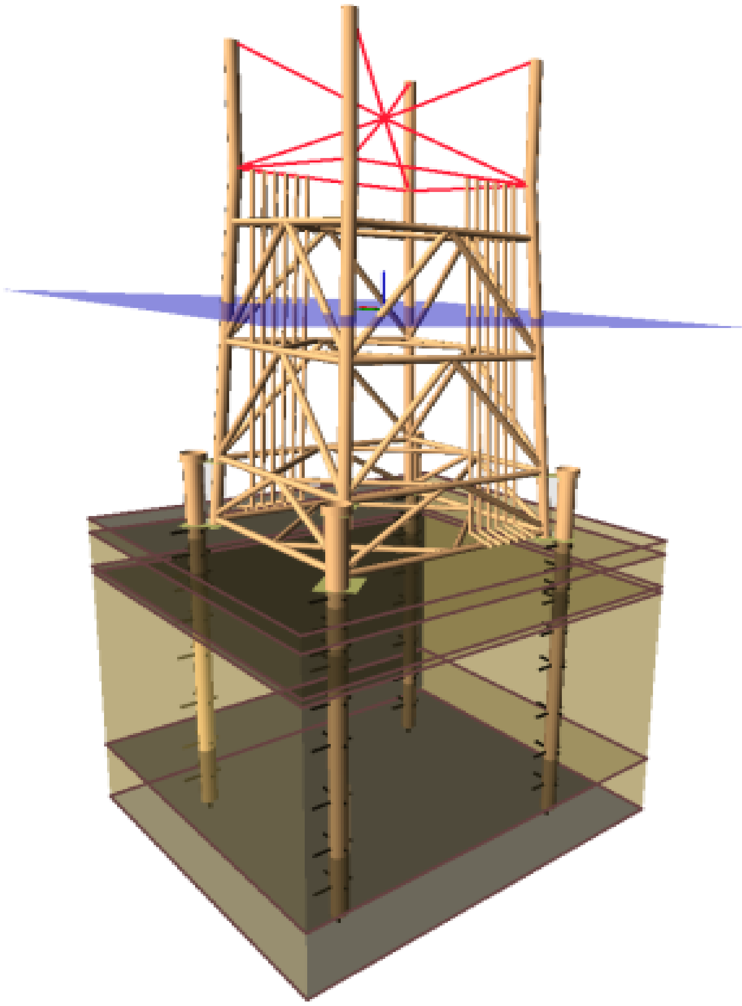


Figure 3-22: Geotechnical model

3.3.1.3 The load modelling for ALS

In the ALS, all kind of forces are considered. They are dead weight of the structures, live load as the weight of the topside, marine growth weight, buoyancy and the environmental loads. They act simultaneously on the structure and strain the carrying capacity of the structure. They are defined as load cases as in Table 3.3-3.

To be kept in mind is that, the dead weight of the structure itself is distributed along the structure and the load varies therefore over the height of the jacket. The fiven value counts as the entire weight, and so do the marine growth weight and the buoyancy loads. The topside weight is represented by a point load acting on the center of gravity of the topside, see Figure 3-22. The wind load is also simplified as a point load acting on the topside central. On the other hand, the wave load is to be calculated comprehensively and in various directions. The same wave is to be applied in three directions referring to different load cases. It is due to the fact that the jacket has a symmetrical shape but not cylindrical. Hence the overall stiffness depends on the lateral load direction. The load cases are linearly combined and it is to be arranged that the wind and current load have the same direction as the wave direction.

Table 3.3-3: Load cases

Nr.	Grundlast / lateral direction		Values / kN
0	Dead weight (jacket + topside)		-39897
1	Marine growth weight in water		-1110
10	Buoyancy		12487
100	Live load		-1962
201	Wave and current load 100-years	0°	-
202		45°	-
203		292,5°	-
301	Wind load 100-years	0°	654,3
302		45°	654,3
303		292,5°	654,3

Environmental conditions are given in Table 3.3-4 (LAT refers to lowest astronomical tide).

Table 3.3-4: Environmental conditions

Parameter		Values
Location		Exclusive economic zone
Water depth		35m in relation to LAT
Sea level	Mean sea level	+1m in relation to LAT
	Highest astronomical tide	+2m in relation to LAT
	Storm surge	+2m
Density	Water	1025kg/m³
	Air	1.25kg/m³
Critical wave	With return period 100 years (non-directional)	Height = 17.5m Period = 13.0s
Current	With return period 100 years (non-directional)	1.20m/s
Marine growth	Height & Thickness	50mm from seabed to +2m LAT
	Density in water	4077,45 kg/m³

Load cases are to be vectorially linear-combined with corresponding factors as in Table 3.3-5. For ALS all factors are equal 1,0. For full-capacity analysis the factor of lateral loads are to be modified to acquire F_{fc}.

Table 3.3-5: Linear combination of loads for ALS

	$G_{1.1}$	Nr.	$G_{1.2}$	Nr.	$G_{1.3}$	Nr.	Q_1	Nr.	E_{wc}	Nr.	E_w	Nr.
330000										201		301
330045	**1.0**	0	**1.0**	1	**1.0**	10	**1.0**	100	**X**	202	**X**	302
330292										203		303

$G_{1.1}$ dead weight load
$G_{1.2}$ marine growth weight load
$G_{1.3}$ hydrostatic buoyancy
Q_1 live load (weight of topside platform)
E_{wc} loads by waves and current
E_w load by wind
x variable load factor for full-capacity analysis

The structural model is constituted in the program ANSYS 16.1. The global analyses are done by ANSYS by applying static load cases on the structure. The wave and current load cases, which are compatible with ANSYS program, are derived beforehand from the calculation of quasi static load caused by wave and current according to the procedure given in [2], presented in Chapter 3.2.2. This quasi static analysis is undertaken in a separate program developed by the Department Offshore Installations.

ANSYS supplies the nominal interfaces induced as stress resultants as well as component stresses. They are used in the member and joint checks as well as the calculation of increased stress and damages at the joint weld seam for FLS. These checks are done in separate analyses, which are carried out by another computing program also developed by the Department Offshore Installations.

3.3.1.4 Load modelling for FLS

For FLS, load modelling is performed by defining individual load cases for each individual wave with means of the quasi static analysis as the ALS. Instead of three wave and current load cases, each wave and current with associated parameter of height, period and direction calls for a load case.

These load cases shall be able to provide one stress range at each location of each joint. A wave is assumed to have sine-curve motion, so does the induced stress. The stress range is defined as the maximum value subtracting the minimum value of the stress. It is the amplitude of the stress variation indeed. Thus stress range is the doubled amplitude of the sine-curve stress variation. The stress amplitude and the representative stress range induced by this wave can be calculated as in following equations, where the middle stress is inconsequential for stress range

$$\sigma = \sigma_{amplitude}.\sin(\frac{2.\pi}{T}t + \varphi) \qquad 3.3\text{-}1$$

$$\Delta\sigma = 2.\sigma_{amplitude} = 2.\sqrt{\sigma_{\varphi}^2 + \sigma_{\varphi+90°}^2} \qquad 3.3\text{-}2$$

$\sigma_{amplitude}$	amplitude of the stress induced
$\Delta\sigma$	representative stress range induced by one wave
σ_{φ}	stress induced at wave at t = 0
$\sigma_{\varphi+90°}$	tress induced at wave at t = T/4 (T is the wave period)

The wave stepping function is therefore used only for two steps of wave, namely at the wave step with φ and (φ + 90°) of the wave motion.

After determining the stress range, the number of its repetition shall be counted. The waves' distribution given in scatter diagram shall be applied to derive the absolute data of waves for 30 years from statistical data. Having the number of stress cycles and the associated stress ranges and SCFs, the damage caused can be ascertained with means of the S-N curve. Combining the single damages over the individual loads cases, cumulative damage can be calculated.

3.3.2 FLS results

As mentioned, the 16 SCFs are afore calculated and recorded. Information needed to select the appropriate SCFs is supplied by the global analysis of ANSYS. The cumulative damages are visualized in colors at each joint of the jacket. Most of the damages are in the tolerable range. There are three damages which exceed the limit of 1,0. They locate all close under the water level. This is understandable because at this water depth, the waves have the largest height and thus derive the greatest stress ranges, see Figure 3-23.

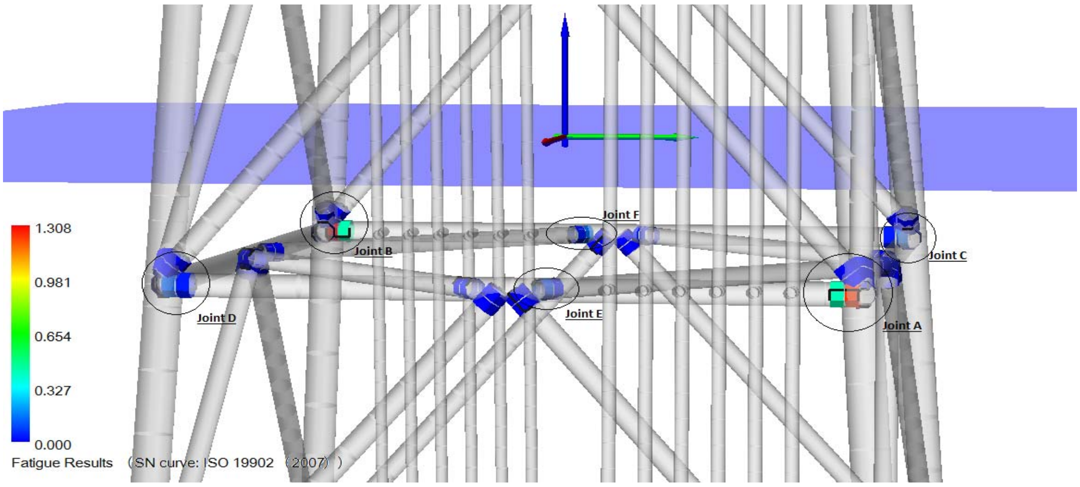


Figure 3-23: Cumulative damage at weld after 30 years, part 1

For the 3 fatigue critical joints, geometrical parameters are listed below. They are all in the validation range, the Efthymiou's equations are therefore verified and the damages calculated account for the correct results. Joints with cumulative damage over 1,0 are to be assigned the highest level of probability of occurrence. They include joint A and B as marked in Figure 3-23. Table 3.3-6 shows the geometrical parameters of A, B and C.

Table 3.3-6: Geometrical parameters of investigated joints

	minimum	Joint A	Joint B	Joint C	*maximum*
beta	0,2	0,667	0,667	0,667	1
gamma	8	18,333	18,333	18,333	32
tau	0,2	0,444	0,444	0,444	1
theta	20	87	87	87	90
alpha	4	22,129	22,129	22,129	40
zeta	0	0,7388	0,7388	0,7388	1

Otherwise, at the deeper level there are several damages which also appear as significant. Although at the deeper level of water, there is no wave load but only the current load, the stress resultant features high values due to the bending component stress which are caused by the lateral load of wave combining the long lever-arm, see Figure 3-24.

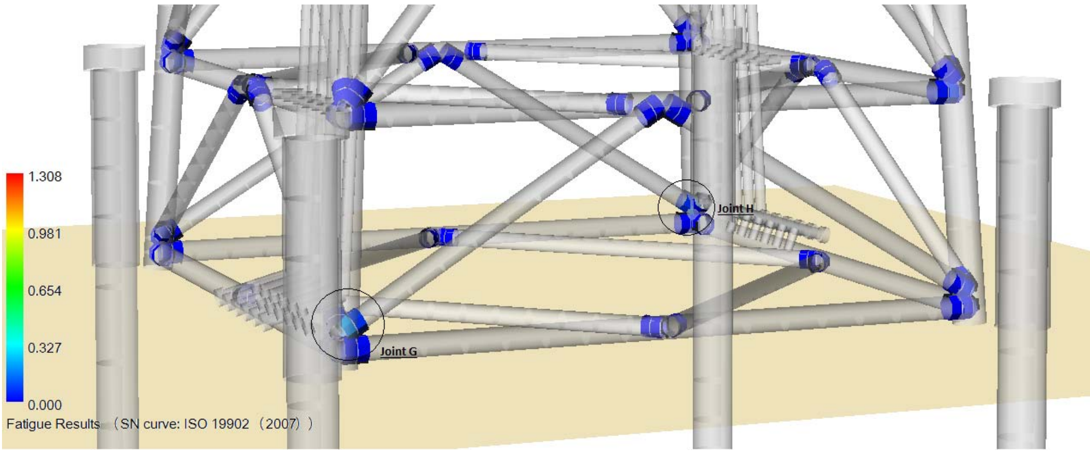


Figure 3-24: Cumulative damage at weld after 30 years, part 2

For each weld seam of a joint, cumulative damages at 8 points on the chord side and 8 points on the brace side are calculated. Both sides can be perceived by two different colours at one joint. It is to be noticed that the damages at chord sides are greater than at the brace side. Accurate damages of joint A and B are listed in Table 3.3-7. The position numbers are consistent with the circumferential points indicated in Figure 3-16.

Table 3.3-7: Accumulated damages after T = 30 years

Joint	Element	Position	Chord damage	Brace damage
A (Type K)	1902	1 & 5	1,252	0,441
		2 & 6	0,326	0,086
		3 & 7	0,000	0,000
		4 & 8	0,355	0,104
B (Type K)	1885	1 & 5	1,308	0,443
		2 & 6	0,336	0,094
		3 & 7	0,000	0,000
		4 & 8	0,347	0,103

As seen in Table 3.3-7, the cumulative damages on the chord sides are about 3 times greater than those on the brace sides at the same circumferential position. Within one side, the cumulative damages at the saddle points are the greatest and at the crown points are the lowest. The damages are point-symmetrical. Opposing points have similar damages.

After acquiring results from FLS, joint A is to be selected for the member importance analysis. The brace element 1902 is to be removed from the original model (Model 0) to generate the damaged jacket (Model 1.2 as in Figure 3-2). Within the scope of member importance analysis, the Model 1.2 is subjected to the full-capacity analysis under the conditions of ALS to determine the $RCR_{1.2}$. The intact jacket-Model 0 is also subjected to these analyses to determine the RCR_0, so that the reduction of the overall resistance due to the failure can be evaluated.

3.3.3 ALS Results

Utilizations of member are actually computed subjected separately to 9 component stresses and combined stresses. However only the maximum of these values is selected as representative for the member utilization and is visualized in one plot.

Table 3.3-8 summarizes the results of member checks and joint checks for all three combination load cases, which differ from each other through the direction of associated wave and current. Only the utilization of the most critical members and joints are listed herein.

The environmental loads applied are modified by the factor X. The X-factors found in Table 3.3-8 result from the full-capacity analysis. With F_{fc} equal 1,485 times of the F_{100}, the utilization of 1,0 is reached on Model 0. With F_{fc} equal 1,425 times of the F_{100}, the utilization of 1,0 is reached on Model 1.2.

Table 3.3-8: Utilizations in comparison

x = RCR	Direction	Member check		Joint check	
		Model 0	Model 1.2	Model 0	Model 1.2
1,485	0°	0,64	0,64	0,86	0,86
	90°	0,71 Figure 3-25	0,93 Figure 3-26	1,00 Figure 3-27	1,08 Figure 3-28
	292,5°	0,68	0,70	0,91	0,90
1,425	0°		0,61		0,82
	90°		0,89		1,00
	292,5°		0,66		0,86

It can be seen that the wave and current in direction of 90° have the most negative impact on the structure. Hence at this combination load case, the maximum utilizations are higher than those in the other combination load case. And it can be noticed that the maximum of joint utilization is always greater than the maximum of member utilization under the same conditions. X implies the RCR defined in Equation 3.1-1 and 3.1-2. As the benchmarking values, the F_{100} are given in Table 3.3-9.

Table 3.3-9: Unfactored environmental loads F_{100} in directions 0°, 90° and 292,5°

LC	Fx [kN]	Fy [kN]	Base shear [kN]
330000	19906,9	0	19906,9
330090	39,7	20887,1	20887,14
330292	7638,2	-19601,1	21036,76

The redundancy ratios RCR as defined in Chapter 3.1.3.2- Consequence of Failure are determined:

- $RCR_0 = 1,485$
- $RCR_{1.2} = 1,425$

Figure 3-25 displays the member utilizations of Model 0, under wave and current load in the direction of 90° combined with all other existing loads with X = 1,485. At this load combination, the members are not working to full capacity, but the joint are, see Figure 3-27. It is also to see that although the model is symmetrical, the utilizations are not. This is because of the J-tubes, which influence the hydrodynamic stream and therefore the load on the structure.

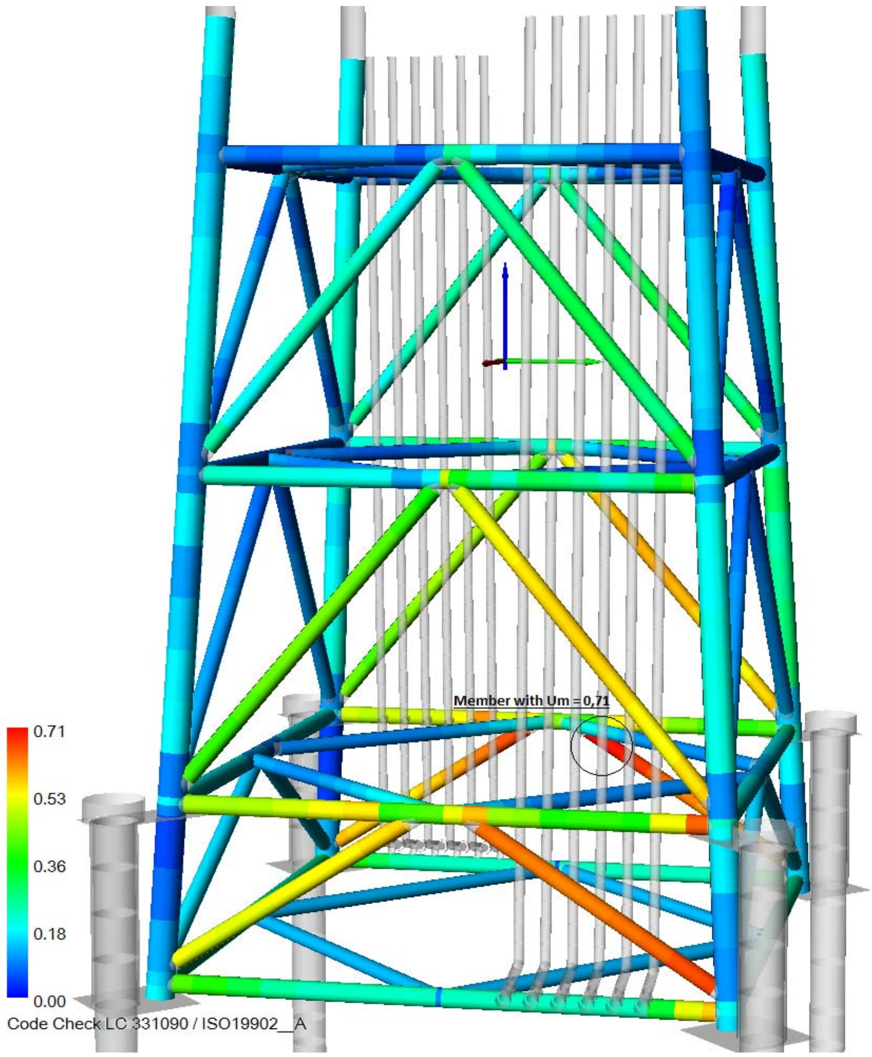


Figure 3-25: Model 0 – member check, load direction 90°, x = 1,485

Figure 3-26 displays the member utilizations of Model 1.2, under wave and current load in the direction of 90° combined with all other existing loads, with X = 1,485.

Model 1.2 is subjected to the same load conditions as Model 0 in Figure 3-25 to clarify the differences. At the position marked with "Removed element", the element 1902 was removed from the jacket, simulating the tearing of the brace. The redistribution of load is clear to be seen due to the changes of member utilization. The member strengths shall be the same, while the local loads vary.

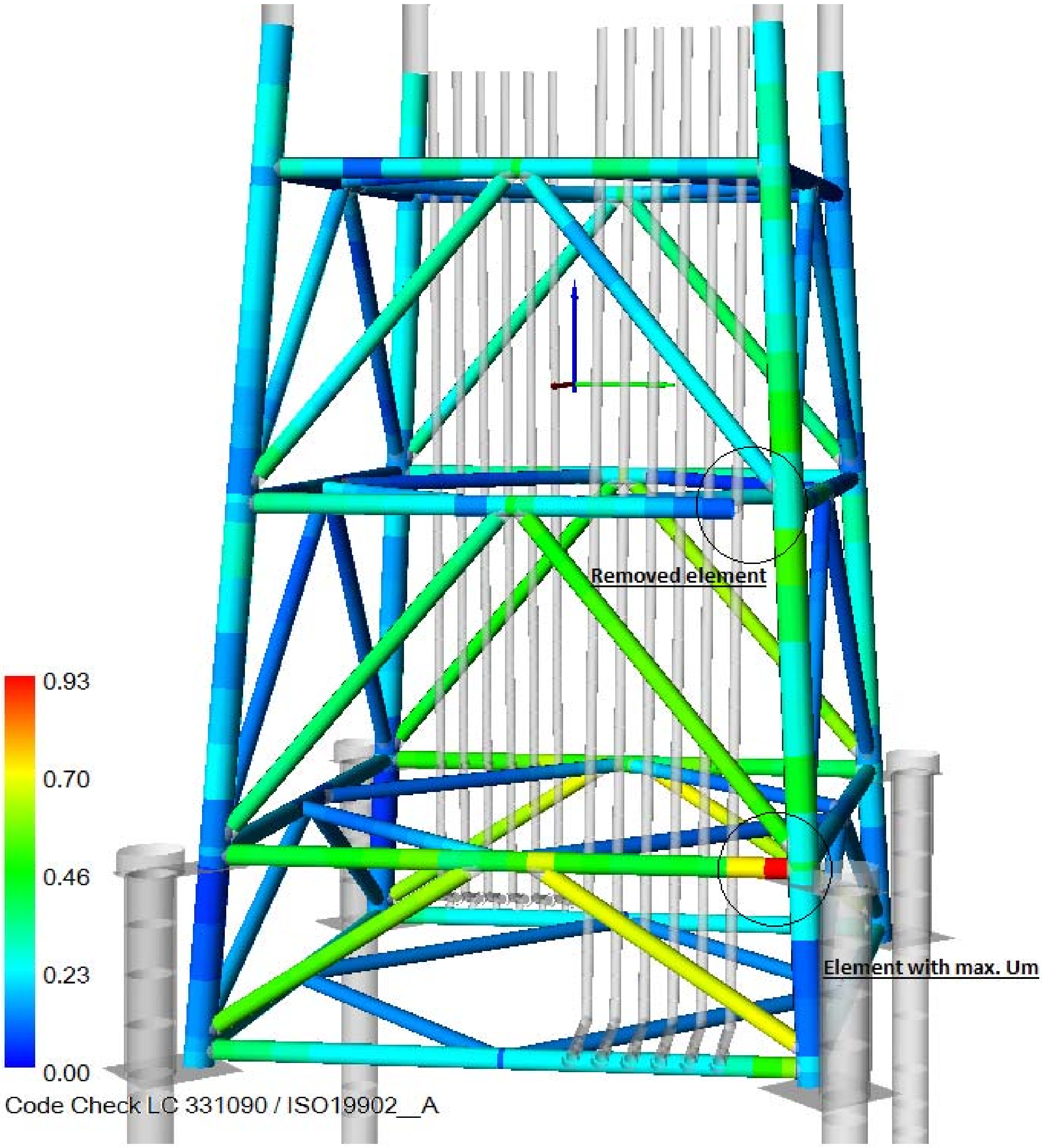


Figure 3-26: Model 1.2 – member check, load direction 90°, x= 1,485

In Model 1.2, another member is maximally utilized. The hydrodynamic component of combined load is in the same direction with the y-axis, the green arrow. This explains the position of the most exhausted member, possibly due to the high compression component stress.

Figure 3-27 displays the joint utilizations of Model 0, under wave and current load in the direction of 90° combined with all other existing loads with X = 1,485. The fully utilized joint is marked with “Joint with Uj = 1,0”.

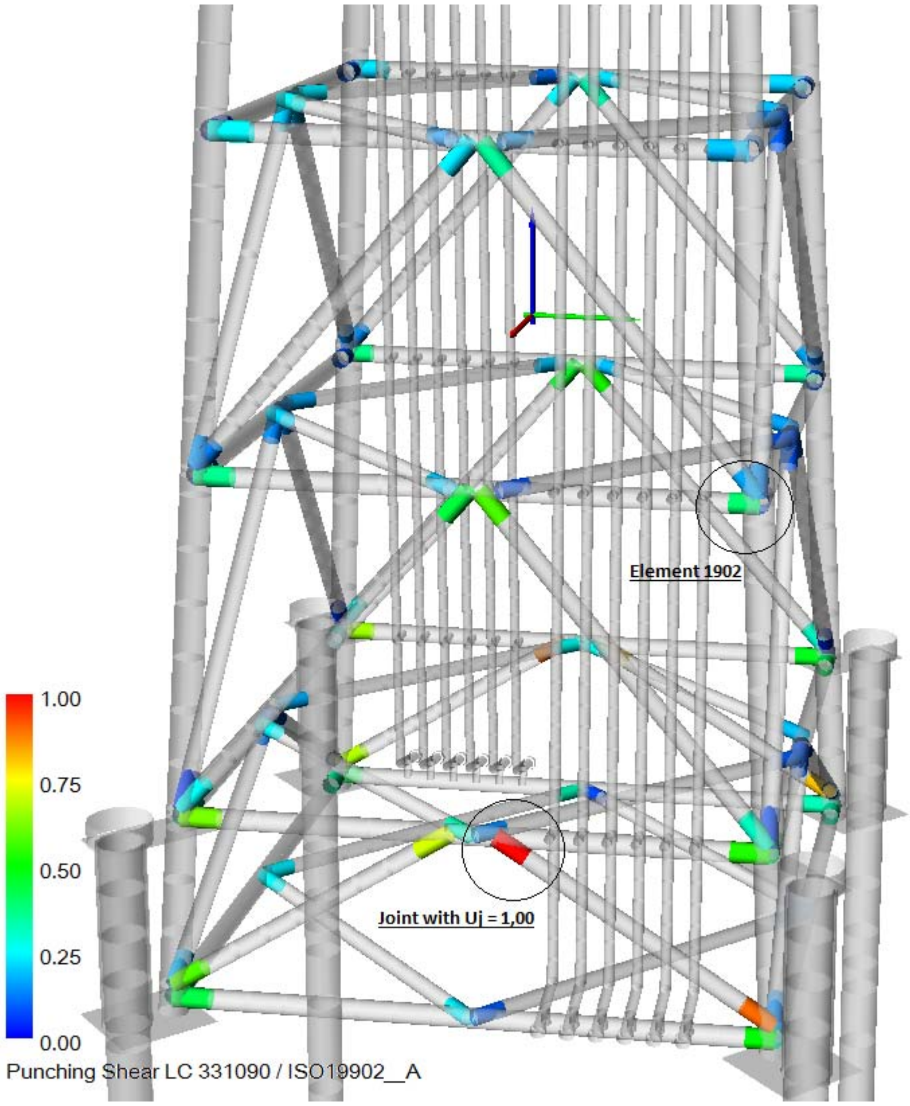


Figure 3-27: Model 0 – joint check, load direction 90°, x = 1,485

The hydrodynamic loads exert in the direction of the green arrow (y-axis). This causes a high axial load on brace of the joint marked. Furthermore the joint is created by a diagonal and a cross struts and the wall thickness is relatively small. The cross strut here serve as chord, and the wall of chord is exposed to the punching shear. Here again the asymmetry of the utilization can be seen.

Figure 3-28 displays the joint utilizations of Model 1.2, under wave and current load in the direction of 90° combined with all other existing loads with X = 1,485. The fully utilized joint is marked with “Joint with Uj = 1,08”. The tearing of the brace is also marked with “Removed element 1902”. The joint, which exhibits the utilization of 1,0 at Model 0, is at Model 1.2 utilized with 1,08. Under the same load conditions, joints of the damaged model are more utilized than those of the intact model.

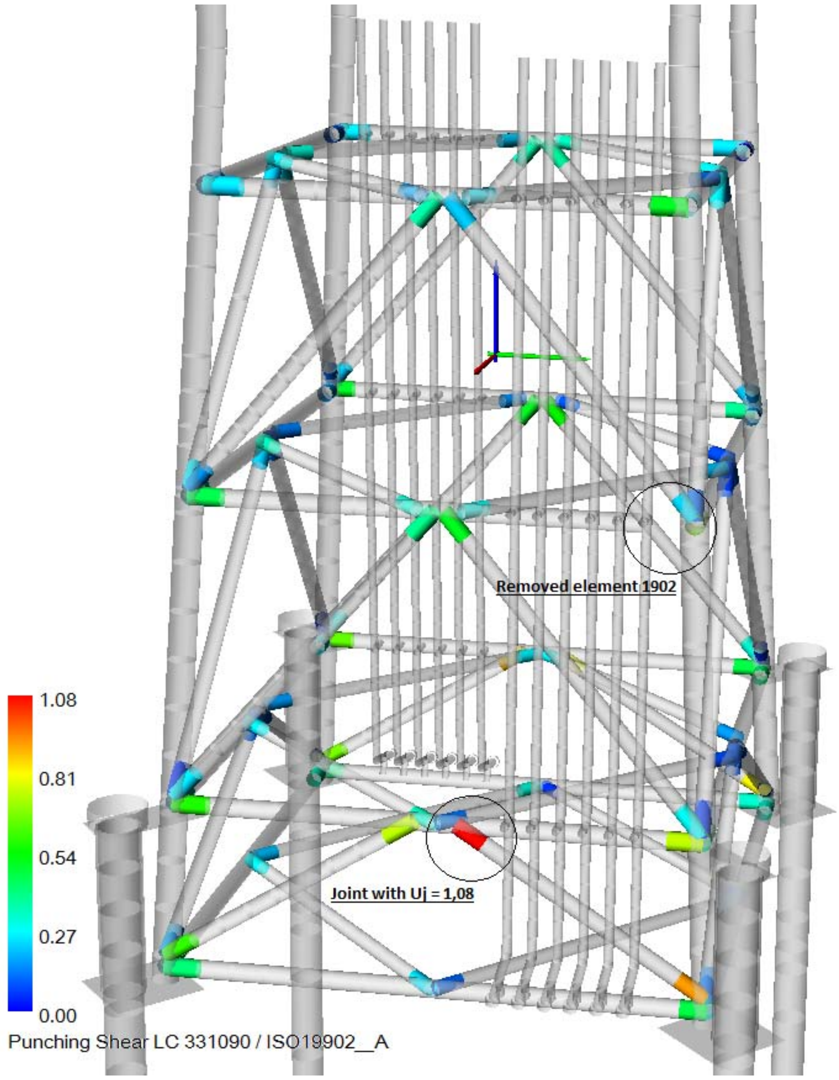


Figure 3-28: Model 1.2 – joint check, load direction 90°, x = 1,485

Because the maximum utilization of Model 1.2 under these load conditions exceed the critical value of 1,0, Model 1.2 is then to be subjected to a lower hydrodynamic loads. The X factor is hereby modified in the full-capacity analysis until the maximum utilization is reduced to 1,0. It is the results of an X of 1,425. With X = 1,425 the jacket features the member and joint utilizations analogously to it with X = 1,485. The values of utilization are reduced insignificantly but evenly over the jacket.

The reduction of X from 1,485 to 1,425 demonstrates the reduction of structural integrity of the structure due to the brace absolute failure. It is to be expressed in the percentage basis as in Chapter 3.1.3.2.

$$a = = \frac{RCR_0 - RCR_{1.2}}{RCR_0} 100 = 4{,}04\% \qquad 3.3\text{-}3$$

The reduction of representative structural resistance of 4% is considered as not significant. Thus the importance of the investigated brace is low and therefore the consequence of its failure can be assigned in the lowest level - CoF level 1.

3.4 Risk Matrix of Structural Elements

As acquired in the results of FLS and ALS, risk assessment of joint A can be accomplished as an example. Joint A belongs to the category of highest probability of occurrence and the lowest consequence of failure.

Table 3.4-1: Risk level of joint A

Probability of Failure	high-4	4	8	12	16
	m.high-3	3	6	9	12
	m.low-2	2	2	6	8
	low-1	1	2	3	4
		low-1	m.low-2	m.high-3	high-4
	Consequence of Failure				

In this manner, risk for all joints can be assessed and a list of joints in the same risk level can be made. Risk levels specify the planning of inspection intervals and eventually preventive measures. Critical points in the sense of risk of fatigue failure can be treated before the failure occurs, e.g. by grinding.

Risk levels can be used for adjustment of the default inspection program according to [2] clause 23. For example, the joints of highest risk levels are to be inspected annually. Other levels can be arranged in longer intervals of 2 years or 5 years.

4 Strategies for Periodic Inspections of Plant Systems of Offshore Manned Topsides

4.1 General

This section deals with another object of periodic inspections – the topside plant systems of a power offshore substation from the perspective of a technically competent independent third party as surveying organisation. This includes auxiliary systems and safety installations, for instance, cooling systems, control and monitoring system, extinguishing systems, etc. On a manned platform, there are also a living quarter and workshops. Beside the main functioning systems of converter and transformer, these supporting facilities bear up the topside asset and allow assuring the proper operation.

With the same target of safety for personnel, asset and environment, the reliability of topside systems shall be maintained in operation as well as during maintenance. Due to the island operation, there would not be immediate external support in critical situations and the possibility of evacuation is constrained. In this context, the importance of safety for personnel is to be enhanced. The intrinsic safety of the platform shall be therefore guaranteed continuously. As a result, safety-relevant systems are primary objects of maintenance and condition monitoring.

This chapter concentrates on the planning of periodic inspection of a water extinguishing system as an example. The PI of safety-relevant systems is one of the regulatory requirements and shall be conducted by or under the supervision of an accredited independent third party as the surveying organisation. The inspection shall be able to report the condition in order to verify the quality of the maintenance and to confirm the fitness for purpose of the system in offshore condition.

In such a plant system, it deals with many different specific components and relates to the design requirements as well as consequential maintenance requirements of the manufacturer for each component. The PI is only a controlling measure. It expects components to be in an operable condition, and assumes therefore the already executed maintenance of system components. In the following chapter, a method of defining the scope of PI is presented for a PI plan which covers essential parts of systems but is reasonable and economical.

4.2 Safety Based Inspections

Similar to the risk base method of PI for a structural system, the strategy of planning PI for a plant system is also conceived by assuming a system failure, evaluating the probability and consequence of this failure, and using these as a base for determining inspection interval and intensity. For a structural system, the failure is the reduction of structural resistance. But for a plant system, the failure is defined as the absolute loss of functioning at a specific place in the asset, although it can be caused by malfunction of one component. In case of an extinguishing system, the failure is the malfunction of system in an application area. The system integrity is not seen as a whole but partially. In the safety based inspection (SBI) the criterion of risk evaluation is not the degradation of the system but the impact of system failure on the personnel and asset.

A water extinguishing system is a safety- relevant system, counted among active fire protection systems. Its proper functioning means safety for personnel and also physical intactness of the asset. The offshore standard DNV-OS-J201 ([4]), which is comprehensively applicable for an offshore substation, outlines the offshore requirements concerning many aspects such as structural design, electrical design, etc. In the chapter "In-service inspection and maintenance", the standard suggests a maintenance concept, which comprises the periodic inspection.

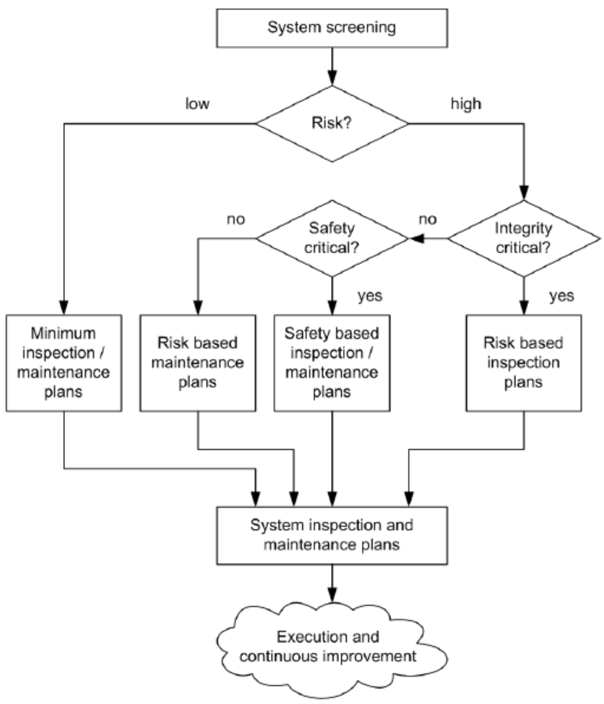


Figure 4-1: Risk based maintenance concept, source [4], P.61

Figure 4-1 displays the scheme of principles for the in-service inspection planning. This aims for keeping the system condition at a level, on which it can work as desired in offshore ambient condition.

4.2.1 Definition

4.2.1.1 Principles of a HAZID study

In the early stage of development, a hazard identification (HAZID) study has to be conducted to investigate external impacts on the platform as well as influences of the platform on the environment. The HAZID study aims for a premature verification and evaluation of design criteria as well as system parameters. The identification and evaluation of hazards in a HAZID study are carried out qualitatively on basis of compliances with requests for health, safety and environment. The study comprises four steps:

- identifying of potential hazards
- investigation of causes
- evaluation of derived risks
- suggestion of constructive and conceptual measures for mitigation or reduction of risks

Not only for development and design, the general principles of HAZID can also be applied in the phase of operation, in maintenance and inspection. Instead of risk mitigation through design, the solution for an installed system can be found through other measures such as preventive maintenance or additional monitoring. Concerning periodic inspections, measures to reduce consequences of failure event are frequent controlling activities in predefined time lags. The inspection intervals depend on the level of risk, to which the investigated area is exposed. For a particular system, the work steps according to principles of HAZID have to be specified accordingly:

- identifying of potential hazards explicitly towards the selected areas and concerning the function of the investigated system
- investigation of causes of postulated events
- evaluation of derived risks in terms of consequence of the events and probability of occurrence of the system failure
- suggestion of measures for reduction of risks with respect to the interval of periodic inspections

The considered system is to be clearly defined to point out the relevant areas on the platform, so that potential hazards can be identified and evaluated for each of them individually. Thereby postulated events due to these hazards shall also be realized so that appropriate countermeasures can be chosen. In this case, a measure is the control of system conditions through inspection. The frequency of the inspection of a particular area is derived from the risk evaluation. Table 4.2-1 display the definition of risk in terms of consequence of failure and the probability of failure occurrence.

Table 4.2-1: Risk matrix for plant system

Probability of Failure	high-1	2	1
	low-2	4	2
		low-2	high-1
	Consequence of Failure		

Risk = PoF x CoF

Failure of an extinguishing system is generally defined as an event of occurring fire which is not eliminated by the firefighting system. Risk of fire failure is defined as the product of probability of occurrence and the consequence of this failure.

For process engineering system, maintenance programs are mostly very strict and precise. Otherwise, the number of areas to be considered is relatively low in comparison with this of a structural system. Hence risks can be defined in a lower number of levels. PoF and CoF are then defined in only two levels. It results in three levels of risk and thus periodic inspections are to be conducted in three different intervals: one year, two year and four year.

While in a structural system safety is defined as the uncharged capacity of strength or the subtraction of load effect from the resistance, in a plant system it is defined as the reciprocal of risk. Therefor the safety level can be much enhanced by reducing the risk. At the stage of operation, risk can be reduced by frequent inspection. Risk assessment within the scope of PI is done with respect to the deterioration over time, the loss of quality of components or the consideration of an error rate in the production and installation of components. It is further described in the following chapters.

4.2.1.2 Consequence of failure

The consequence of a fire failure is evaluated in terms of its impact on the staff, asset and the environment. However they are not separately considered for staff, asset and environment but in combination. Safety for personnel has the highest priority. The areas where the staff is often located shall be ranged as having the highest consequence of fire. For a manned platform, overnight possibility is given and a living quarter is installed. Other areas where the staff may stay in a period of time for controlling, repair or other service shall be taken into account. Many areas are equipped with more than one extinguishing system, for example inert gas or foam in addition to water. But inert gas is only permitted for equipment room because it disturbs the breathing. Some areas therefore rely totally on the water system.

The BSH pays special attention to the effect of fire or explosion on the supporting structure in the topside of the platform. Despite from the civil construction, such offshore structure is built mainly from steel and for the most part large scale. And steel materials display similar behavior when exposed to fire. Figure 4-2 displays the material properties of steel in dependence of the exposure temperature.

Decks and floor of topside are constructed by steel plates reinforced with beams. In case of an increased exposure temperature, two effects are provoked: the reduction of strength and the expansion of the material. The reduction of strength affects especially the load carrying parts, amongst others, the floor under concentrated load. The expansion of material under end constraint leads to the increased inertial stress and charge the structural capacity in addition to the nominal stress. If these two superpose, it may cause fracture propagating to failure of substructure and further to the structure collapse. The topside is therefore usually equipped with structural fire protection. It is the insulation of walls and decks to hold up the spreading of heat, smoke and fire. This is a passive fire protection measure which can be effective up to 60 minutes and the temperature shall not exceed 140°C over the ambient temperature. Within this time, if the fire is eliminated, the system is considered cured. But if the active fire protection fails and the fire load is high enough, the temperature may continuously rise.

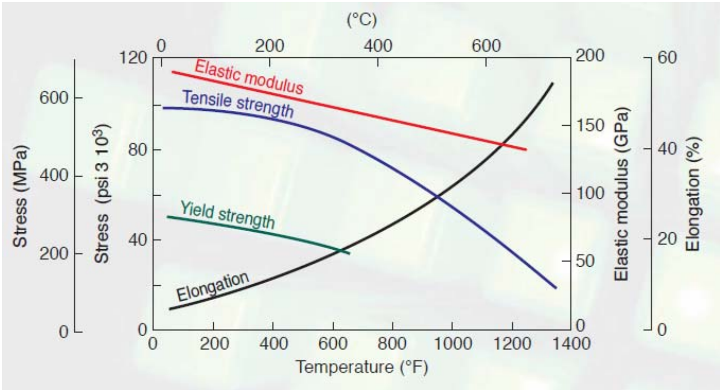


Figure 4-2: Effect of temperature on mechanical properties of a carbon steel, source ISBN No. 0-13-227271-7 (Figure 2.9)

In such a situation, the local structure is endangered. Similar to the jacket, local failure causes load redistribution in the system and designed resistance can be exceeded. This would be sequential failures which, in word case, bring the whole topside to collapse. The fire load shall be considered as a criterion to estimate the consequence of fire.

4.2.1.3 Probability of failure

Analogous to risk assessment for structural system in chapter 3, the probability of failure (PoF) in this simplified method is not quantified as a statistical probability of occurrence of a random variable. The PoF is qualitatively assigned to levels based on the type and the degree of hazards for each area.

Probability of failure occurrence is assessed by the collective contribution of different factors. The probability of fire occurrence can be roughly rated in terms of the degree of danger concerning possibility of ignition and also the fire load. The BSH suggests for instance some of the hazardous areas as followed:

> *"Gefahrbereiche sind Bereiche, in deren unmittelbarer Umgebung besondere Gefährdungen auftreten, die zusätzliche Schutzmaßnahmen zur Risikoverminderung oder zur Begrenzung von Schadensfolgen erfordern. Die Priorität der Schutzmaßnahmen gilt dem Arbeits- und Personenschutz. Darüber hinaus ist auch der Umwelt- und Sachschutz zu berücksichtigen. Typische Gefahrbereiche von Offshore-Bauwerken sind z.B. elektrische Schaltanlagen und Installationen, Bereiche mit erhöhten Brandlasten oder –Risiken (Tanksysteme, Lagertanks, Öltransformatoren), explosionsgefährdete Bereiche, …"* (BSH Standard [1], P.129)

According to this definition of the BSH, hazardous areas are locations at which hazards can occur. For these areas additional measures for mitigation of risk or limitation of the consequential loss are required. First and foremost protective measures should be conducted with respect to safety of personnel. In addition protection of the environment and the asset should be taken into account. Typical hazardous areas of an offshore platform are for example electric switchboards and electric installations, areas with high fire load or explosive areas (ex-zones), etc.

If there is any workshop for metal working and welding, the probability of ignition due to metal shavings and sparks shall be taken into account. Storage of combustible goods like paint or garbage shall be considered either. On the other hand, to be kept in mind are hazards of malfunction concerning technical matters. Failure of components can occur if the system is exposed to aggressive corrosive atmosphere or electric sparks.

4.2.2 Scope and Report

The risk assessment helps determining the level of safety requirements, to orientate the focus of the periodic inspection with respect to inspection interval and inspection intensity. However, the inspection plan in detail shall be defined based on the compositions and characteristics of the system.

The required surveillance report shall cover results of the periodic inspection, which give an overview of the system's physical and functional state. The inspection is performed with the assumption that maintenance is carried out in compliance with the manufacturer's instruction. Due to the restricted conditions of offshore location, maintenance is mostly carried out monthly while it normally requires a weekly inspection. Therefore the actual inspection program shall be enhanced to compensate this nonconformity. Tests which may be carried out for the PI by surveyor:

- Review of the maintenance diaries
- Visual inspection
- Function inspection and others

Apart from the system documentation the maintenance documentation supplies information about the system in the past operating period. An overview of the general condition of the system and results of the prior inspections can be acquired. They serve as references for planning and preparation of the coming inspection. It contributes a smooth process of determining date, ordering test equipment and most importantly inviting the accredited expert and authorities if required.

The visual inspection shall be performed to control the general states of the components and to check whether there is any physical damage. Possible findings would be such as corrosion, leakage or mechanical damages. Corrosion for example shall be prevented especially at the openings of hydrants. Equipment support structures shall be checked for deformation or damages, as well as excessive corrosion.

The function tests are to comprehensively control the functionality of equipment such as pumps, sensors, and valves. The equipment shall work in the desired manner according to the approved system specification and the cause and effect matrix. This may be tested in combination with the fire detection system and its automation. For this purpose scenarios of fire may be presented.

4.3 Water Extinguishing System

A water extinguishing system is one of the active fire protection systems or firefighting systems. It is a system consisting of several units, from extinguishing pumps, pipelines, hydrants through to the sprinkler heads. All of these subsystems use the same source of water and pressure supplier. For the automatic function a lot of mechanical and electrical components as well as sensor or other automation components are required. The system is not in permanent operation, but in case of an emergency and it requires a readiness for use of 100 percent.

Until now, most firefighting systems are designed in accordance with standards and rules of VdS (VdS Schadenverhütung GmbH- a subsidiary of GDV, the General Association of the German Insurance Industry), which is accredited for verification and certification of fire protection equipment. In case of offshore application, additional measures shall be performed corresponding with the offshore specific requirements.

4.3.1 System Description and Application Areas

The topside is not equipped with water extinguishing system in all of its locations. For certain areas, such as converter room or reactor room, inert gas is in use exclusively or with other systems in addition. This is decided by the suitability of extinguishing with water. For fire of oil or fuel, inert gas or foam shall be used. Figure 4-3 shows the block diagram of a sample water extinguishing system.

A water extinguishing system consists of a water source, extinguishing pumps, pressure-maintaining pumps, pipelines, sprinkler- or deluge heads, automation installation. To protect the pipelines against freezing and to have the water ready, the water is only pumped to a certain point. The rest of the system is kept dry.

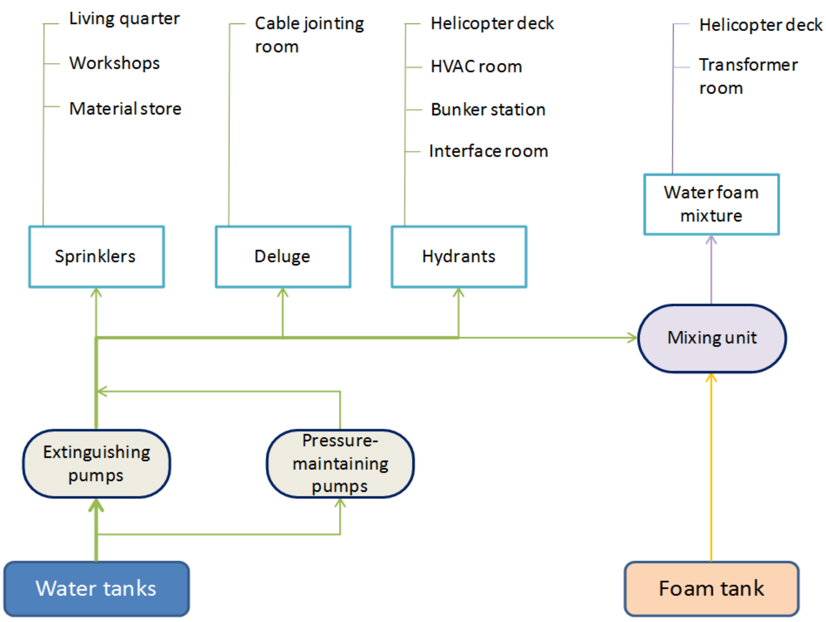


Figure 4-3: Block diagram of a water extinguishing system

The water extinguishing system is installed in the areas listed above, for those hazards shall be identified and risks shall be evaluated.

For inspections of an extinguishing system, the documentation of the system and the associated maintenance program as well as maintenance documentation shall be provided generally. Concerning the system, they include the system description, process flow diagram, piping and instrumenting diagram, cause and effect matrix and operational manual if available. For more specified information isometric plan of the system may be required. It facilitates the localization of the piping for inspection. Concerning components, they include type-certificates, if applicable, manufacturer instruction, and maintenance diaries. The documentation shall be in the up-to-date status. Otherwise, documentation of non-conformities or technical amendment shall be provided if applicable.

4.3.2 Risk Assessment

4.3.2.1 General

The main test object of periodic inspections is deterioration of the system over time. Hence the hazards to be considered are only those which damage the system gradually and events which occur accidentally. Table 4.3-1 lists the 4 main hazard categories, associated postulated events which have negative influences on the extinguishing system or facilitate fire, the impacted areas and the suggestion for inspections.

Table 4.3-1: Application of HAZID principles for water extinguishing system

Hazard categories	Postulated events	Impacted areas	Inspection of extinguishing system
1. Corrosion	Corroded components	All	Visual inspection (annual)
2. Mechanical hazards	Loss of supporting capacity Leakage of pipes or tanks Loss of tank content medium	All	Visual inspection (annual)
3. Electrical hazards	Electric sparks, ignition	HVAC room Cable jointing room	Function inspection (risk based)
4. Fire load	Prolonged fire	Transformer room Material store	Function inspection (risk based)

Corrosive atmosphere is one of the offshore specific environmental conditions, which impairs the materials and causes deficiencies of physical conditions of the system. Apart from steel supporting structures, steel equipment and piping, small to micro electrical parts, such as sensors or switchboards, are strongly exposed. Corrosion at important signal conducts or connections can result in malfunctions and thus functional loss of the overall system. The ingress protection (IP code) or tightness of control cabinets shall be adequately given and maintained. Despite correct IP, in the framework of visual inspection, current condition of those parts should be covered.

Mechanical damages can be caused by various reasons, for instance, loosening of bolts, dynamic loading, pressure shocks, and collision during other operations, etc. Leakage of medium is to be considered.

Electrical hazard implies in this circumstance the possibility of electric sparks and considered as a source of ignition for fire. Also an increased temperature should be taken into account. Areas with high electrical performance and especially high voltage shall be taken into account.

Fire load is the most critical hazard for fire. It implies the amount of flammable material which may feed the fire. In the transformer room there is oil available, and in material store it can be different substances.

4.3.2.2 CoF assessment

As defined above, the consequence of fire in all areas can be assessed in terms of degree of personnel harm and structural damage adjacent to the room and follow-up impact on the overall topside structure. With means of a structural analysis, the areas with high structural utilization can be detected (concentrated load: weight load of equipment, main load bearing structures, adjacent to legs). Substructures under high load have to be considered in combination with the fire load exposed. If a structural analysis of the topside is not given, CoF can be estimated solely in terms of criticality of failure for safety of personnel as follows:

- *Low*: minor damage, local damage, no injuries or loss of life (Cable jointing room, helicopter deck, interface rooms, bunker station, HVAC room)
- *High:* structural resistance fails, injuries or loss of life (living quarter, workshops, interface room, transformer room, etc.)

4.3.2.3 PoF assessment

To evaluate the probability of a fire event, causes of fire shall firstly be identified. With regards to source of ignition and fire load, the areas can be assigned into 2 levels of probability of occurrence.

- *Low*: no source of ignition, no combustible goods (living quarter, bunker station, interface room, cable jointing room)
- *High*: source of ignition available and combustible goods available (transformer room, workshops, material store, HVAC room)

Apart from these main criteria, other factors concerning the system itself should be taken into account either. It raises the questions: what could lead to failure; what is the source of error; and how probably could it happen? Probability of failure is influenced by such factors as:

- Compliance with maintenance plan
- Restricted/open accessibility
- Failure probability in production
- Exposed to heat, pressure, radiation, salty air, explosion,
- Condition based findings
- Component history

4.3.2.4 Inspection intervals

According to the risk matrix there are 3 levels of safety, to which the areas can be assigned. For areas with risk level 1 PI shall be performed annually, risk level 2 once every two years and risk level 4 once every 4 years. Mutual equipment such as pumps and a part of the piping are therefore inspected in the shortest interval.

Table 4.3-2: Risk levels of the topside areas

Probability of Failure	high-1	_HVAC room _material store	_transformer room _workshops
	low-2	_cable jointing room _helicopter deck _bunker station	_interface room _living quarter
		low-2	high-1
	Consequence of Failure		

It would be insufficient to inspect solely the extinguishing system, because its function is in the tight cooperation with fire detecting, alarming and controlling systems. The PI is thus to be performed in connection with these systems and can confirm their condition partially at the same time.

The inspection plan for PI has to be arranged in a way so that the tested articles are associated and assigned to spaces or rooms.

4.3.3 Scope of Periodic Inspections of Water Extinguishing Systems

Concerning the requirements for the fire extinguishing system, the offshore standards DNV-OS-J201 ([4]) refers to appropriate and applicable issues from the MODU code 9.4 (Mobile offshore drilling unit) and the SOLAS regulation II-2 (Safety of life at sea). Both of these regulations are developed and applied in the maritime industry. Although our sample converter platform is a fixed installation, it shows a certain similarity to ship operation with respect to safety at sea and the requirements of island operation. Also developed for ships and mobile units, the MSC.1/Circ.1432 (Maritime Safety Committee.1/Circular 1432) is the “Revised guidelines for the maintenance and inspection of fire protection systems and appliances” ([16]). It also contains the requirements for maintenance which correspond to the design and operation of such installations according to MODU code and SOLAS. MSC.1/Circ.1432 can be therefore applied for the PI of a water extinguishing system on an offshore substation.

However, the standard of IMO is a comprehensive guideline for general maintenance. It requires testing and inspections weekly, monthly, etc. The PI is after all a controlling measure and its scope shall be confined, so that it can prove the satisfaction of these requirements expressively but reasonably. The PI includes also issues from the design requirements for system parameter or function to prove the operational readiness.

4.3.3.1 Review of maintenance documentation

In the process engineering, most of devices and apparatus are specified for particular functions and require inspection plans and activities according to the manufacturer’s instruction. Furthermore, the operator has also the maintenance plan depending on the own criteria for safety and functionality, as well as the company’s priority and philosophy. A sophisticated maintenance plan and its execution are dedicated to the operator. But the operator is committed to the periodic inspection. To achieve good results of the periodic inspection, test objects shall be in proper conditions and this presupposes as usual the regular maintaining. If there is any non-conformity or technical amendment, relevant components shall be checked as in in-commissioning inspections. For those which are type-approved, the validation date of certificates shall be checked.

Reviewing the maintenance documentation supplies an overview over the general condition, information as well as the amendment history of the system.

4.3.3.2 The visual inspection

Thorough examinations of the exterior conditions, mainly with respect to corrosion and mechanical damage, are conducted by visual inspections. They are to be carried out for all visible and accessible parts of the systems. It includes herein also the indication of system parameters and the position of valves according to piping and instrumentation diagram (P&ID) of the system.

Table 4.3-3: Visual inspections acc. to [16] for water extinguishing systems

Unit/ Room	Tested articles	Verification of
Fire pumps and tanks	Support structure and fittings	corrosion or damage
	All accessible components	in the proper condition
	Fire hydrants, hose and nozzle	in place, properly arranged, and serviceable condition
	Hydrant valves	in the operable condition
	Level indicator, leakage	correct level of water
Visible pipelines	Support structure and fittings	corrosion or damage
	Pipelines	corrosion or damage, leakage
Foam mixture unit	Control and section valves	in proper open/closed position
	Pressure gauges	in the proper range
	Foam concentrate	in the proper quantity
	All accessible components	in the proper condition
	Nozzles	clear from debris
Water mist, Water spray, Sprinkler systems	Control and section valves	in the proper open/closed position
	All accessible components	in the proper condition
	High pressure cylinders	external examination for evidence of damage or corrosion
		check the date of hydrostatic test
	Filters/strainers	clear from debris

4.3.3.3 The function test

"The function tests" is used as caption for this chapter, which actually concerns also flow test, pressure test, etc. They are performed for areas and in intervals as defined according to the risk assessment. The tests mostly refer not only the extinguishing system but also the interfaces between cooperating systems. It comprises the water extinguishing system, fire detection system and the automation of them. Table 4.3-4 shows inspections needed concerning process parameters and procedural functionality.

Table 4.3-4: Inspections acc. to [16] for water extinguishing systems

Unit/ Room	Test type	Tested articles	Verification of
Fire pumps, hydrants, hoses, nozzles	Flow test	pumps	proper pressure and capacity
		isolation valves, hydrants valves,	proper function
	Pressure test	a sample of fire hose at maximum pressure	proper function
	Function test	relief valves	properly set
Fire detection, fire alarm	Function test	all system used to release extinguishing	proper function
		emergency power supply system	proper function
		audible alarm	proper function
Foam mixture unit	Flow test	water supply, foam pumps	proper pressure and capacity
		piping	after service thoroughly flushed, blow dry with compressed air or nitrogen (remove debris, contamination)
		cross connections	other source of water supply given
	Function test	pump relief valves	properly set
		control/section valves	in correct positions
	Foam test	a sample of foam concentrate	
Water mist, Water spray, Sprinkler systems (in sections)	Function test	all water mist, spray, sprinkler	proper function using test valves
		pumps	proper pressure and capacity
		cross connection	proper operation
		relief valves	proper set
		control/section valves	correct position
		emergency power supply switchover	proper function
		piping	
		automatic sprinkler/water mist nozzle (minimum 2)	proper operation
	Flow test	open head water mist system*	water through the nozzle

A water flow test is to check the available water supply from the same source to all the areas equipped. The measurement in time and pressure verify the ability of the pumps and if the valves are in correct position, so that the water is obtainable at required areas, all hydrants and provided for foam-water mixing unit. This may also prove if the pipelines are free or if they are blocked. Along with it, the function of indication system can be checked as well. Flow test is important for a dry pipe sprinkler system. Because the period it need for the water to reach the extinguished area should be under the required maximum time of water delivery.

The pressure test as its name is to check if the fire hose can withstand the pressure required in fire-fighting operations. Hoses shall be stable to provide enough pressure for strong water jets.

In a function test, the functioning of all components and especially the automation of system is checked. They shall perform and appear in operations in accordance with the cause and effect matrix. Mostly, false signal is sent out to activate the extinguishing activities comprising pumps, valves and alarm system. To check the detection system, false fire or smoke is given, but the water flow is redirected back to the tanks. The emergency power supply system is also to be checked in the function test.

The foam test refers to the laboratory analysis of the foam quality. It includes the analysis of physical properties, such as pH or density, expansion and drain time, etc.

4.3.3.4 Special requirements for 5-yearss tests

To be taken into account is the 5 year-tests. They are applied for foam mixture unit and the sprinkler as well as water mist and water spray system.

The internal inspection refers to the test, which requires the system to be out of order and be disassembled for proper examination from inside the equipment. It includes inspecting for corrosion and wear around welded seams, nozzles and vessel connections. The corrosion in process engineering implies also the microbiologically-induced corrosion. Non-destructive test can be used in lieu of opening the piping. Dry and wet valves and piping may require different tests.

In addition to the foam quality check, the mixing ratio of the foam mixture unit shall be assured. For location without electrical access, batteries supply energy for the operation of valves. They shall be also checked at least once in five years, if available.

Table 4.3-5: Five-year tests acc. to [16] for water extinguishing systems

Unit/ Room	Test type	Tested articles	Verification of
Foam mixture unit	Internal inspection	all control valves	
	Flushing	piping with water	
		drain and purge with air	
	Visual test	all nozzle	clear of debris
	Foam test	mixing ratio	
		foam properties	
Water mist, Water spray, Sprinkler systems (in sections)	Internal inspection	all control/section valves	
	Flushing	piping with water	
		drain and purge with air	
	Condition check	batteries	

The function inspection 5-year and inspections shall be conducted by authorities of corresponding technical fields. And for all inspection, documentary proof shall be given.

5 Conclusion

The objective of this thesis has been to suggest a conceptual and methodical solution for the planning periodic inspections of an offshore power substation. The basic principles of the risk based approach were presented and interpreted to declare its suitability for use to be applied for planning periodic inspections of structural systems as well as plant systems. Following this, the methodology and the application were described individually for a sample offshore jacket, as a structural system, and a sample water extinguishing system, as a plant system. Although the risk parameters are differently defined for each system, the risk assessments for both of them resulted in a transparent basis for the determination of inspection scopes and intervals.

Chapter 1 provided an overview of an offshore power substation and the needs as well as the obligation of periodic inspection of such a platform. In chapter 2, the periodic inspection was presented as the compulsory controlling measure of the platform condition in the operating phase required by the licensing authority. Here also Rules and guidelines to be applied were chosen with regard to the expertise needed and the permitted level of requirements. Chapter 3 and chapter 4 refer to the methodology adapted to a structural system and a plant system in detail, and the application for the sample systems.

Chapter 3 described the principles of a semi-quantitative risk-based approach, definitions of risk parameters in terms of structural characteristic values including the basis and procedure for their calculation. Furthermore, risk assessment and calculation of an offshore jacket was exercised subsequently. It concerns herein the assignment of tubular joints to levels of probability of occurrence due to their fatigue strength, and the assignment of these joints to levels of consequence of failure due to the importance of the member related to the joints. The member importance is estimated with means of a series of structural analyses.

In contrast to a structural system, it is difficult to evaluate risk for a plant system with mathematical or statistical methods. The risk assessment for a plant system in chapter 4 was therefore qualitatively conducted based on the general principles of a hazard identification study. For a water extinguishing system, risk parameters were defined in terms of various hazard categories to the system and the consequence of the system failure to personnel, assets and the environment. Due to the characteristics of these exposures, different inspections were suggested according to the guidelines for maintenance of a water extinguishing system on ships.

The simplified risk-based methods as introduced in this thesis featured many advantages for the practice of periodic inspections. Although risk-based methods are systematic methods which supply essential and traceable tools for consideration of planning inspections, it is however only a suggestion. Whether the methods are to be applied and to which extent the inspections shall be performed are questions for the operators. The operators are not only responsible for the safety of their staff and the platform; they are also in charge of production and business. The risk-based and safety-based evaluation was carried out based on many assumptions and neglects in the calculation as well as in manufacturing, transporting and installing phases of the platform. The planning periodic inspection shall therefore consider the on-site information and findings, experience factors or unusual weather in a long term view.

Nevertheless the risk-based and safety-based approaches can be applied in an adjustable way. They can be refined and enhanced as the case demands. It will require more research and elaboration and involve more technique. But the results can be very reliable and thus contribute to a successful and effective inspection program.

Literatures and Sources

[1] Standards Bundesamt für Seeschifffahrt und Hydrographie 2015, *Mindestanforderungen an die konstruktive Ausführung von Offshore-Bauwerken in der ausschließlichen Wirtschaftszone (1. Fortschreibung)*, BSH Standard für Konstruktion. [28.07.2015]

[2] Standards International 2007, *Petroleum and Natural Gas Industry* - *Fixed steel offshore structure,* ISO 19902:2007, International Organization for Standardization, ISO.

[3] DNV GL AS, Oil & Gas Germany, Offshore Installations 2015, *Vergleichsstudie zwischen BSH und ISO 19902, Report nr. GLO-15-794,* DNV GL AS Report. Available from: DNV GL, Oil & Gas, Offshore Installations. [11.06.2015].

[4] Det Norske Veritas 2009, *Offshore Substations for Wind Farms*, DNV OS J201, Standards Det Norske Veritas, DNV.

[5] International Institute of Welding 2008, *Recommendations for Fatigue Design and Welded Joints and Components*, IIW Document IIW-1823-07 ex XIII-2151r4-07/XV-1254r4-07, IIW.

[6] DNV GL AS, Oil & Gas Denmark, Structures, Pipelines & Risk 2014, *Risk based Inspection Planning Methodology for Jacket Structures, Report nr. OEEDK-201423140, Rev.0,* DNV GL AS Report. Available from: DNV GL, Oil & Gas, Offshore Installations. [June 2015].

[7] Harold V. Thurman and Alan P. Trujillo, *Essentials of Oceanography,* 7th edn., Prentice Hall 2001.

[8] American Bureau of Shipping 2003, *Guide for Fatigue Assessment of Offshore Structures,* ABS Houston.

[9] M. Efthymiou, *Development of SCF Formulae and Generalised Influence Functions for Use in Fatigue Analysis,* Schell International Petroleum Maatschappij B.V.

[10] Lloyd's Register of Shipping 1997, *Stress Concentration Factors for Simple Tubular Joints, Report nr. OTH354, HSE Books,* Health and Safety Executive, UK.

[11] Darell F. Socie 2010, *Fatigue of Welds,* Presentation, available from www.efatigue.com, [12.08.2015].

[12] G. Clause, E. Lehmann & C. Östergaard, *Offshore Structures vol. 1, Conceptual Design and Hydromechanics. ,* Springler-Verlag, London Limited 1994.

[13] G. Clause, E. Lehmann & C. Östergaard, *Offshore Structures vol. 2, Strength and Safety for Structural Design,* Springler-Verlag, London Limited 1994.

[14] Mj. Tomlinson, *Foundation Design and Construction, 7th edn.,* Prentice Hall 2001.

[15] Mathew P. Stephens, *Productivity and Reliability Based Maintenance Management,* Purdue University Press 2010.

[16] International Maritime Organisation 2012*, Revised Guidelines for the Maintenance and Inspection of Fire Protection Systems and Appliances*, MSC.1/Circ.1432, IMO.

List of Appendizes

Appendix 01

Time schedule for offshore substations acc. to BSH standard

[1] (*Mindestanforderungen an die konstruktive Ausführung von Offshore-Bauwerken in der ausschließlichen Wirtschaftszone (1. Fortschreibung)*, BSH Standard für Konstruktion. [28.07.2015]), Pages 24-26.

Phase	Ziele und Maßnahmen	Kapitel	Vom Genehmigungsinhaber zu erstellende Unterlagen, die beim BSH einzureichen sind	Vom Prüfbeauftragten zu erstellende Unterlagen, die beim BSH einzureichen sind	Zulassungs-behörde
Entwicklung	Erstellung von Grundlagendokumenten für die grundlegende Konzeption (funktionale Beschreibung) der Offshore-Station im Hinblick auf Nutzung, Sicherheit und Meeresumweltschutz	5.1		Prüfbericht (auf Basis der Grundlagendokumente)	Plausibilitätsprüfung 1. Freigabe (ggf. mit Maßgaben)
	Zusammenstellung der standortspezifischen Daten Standortbewertung für die Gründungsstruktur	1.3.2 2.2 3.1 3.2	Geologischer Bericht (sofern gemäß Standard Baugrunderkundung gefordert) Baugrunduntersuchungsbericht Baugrund- und Gründungsgutachten Meteorologische und ozeanographische Gutachten	Prüfbericht und Konformitätsbescheinigung	
	Festlegung der Entwurfsgrundlagen	2.2 5.2	Entwurfsgrundlage (Design Basis) • Festlegung der Normenhierarchie • Beschreibung der funktionalen Spezifikationen und Anforderungen an primäre und sekundäre Tragstrukturen		
	Vorentwurf der Tragstruktur (einschl. einer belastbaren Abmessung der vorgesehenen Gründungsstrukturen für die UVS)	2.2	Vorentwurf der Tragstruktur	Prüfbericht (Stellungnahme)	
Konstruktion	Fortschreibung von Grundlagendokumenten in Form von detaillierteren Konzepten: • Lasthandhabungskonzept • Brand- und Explosionsschutzkonzept • Zugangskonzept • Raumnutzungsplan	5.3.1		Prüfbericht (auf Basis der Grundlagendokumente)	Plausibilitätsprüfung 2. Freigabe (ggf. mit Maßgaben)
	Fortschreibung der standortspezifischen Daten	3.3	Baugrunduntersuchungsbericht Baugrund- und Gründungsgutachten Geotechnischer Entwurfsbericht ggf. Bericht zur Ausführung der dynamischen Pfahlprobebelastung ggf. Bestätigung der Machbarkeit der dynamischen Pfahlprobebelastungen	Prüfbericht und Konformitätsbescheinigung	
	Fortschreibung der Entwurfsgrundlagen	2.3	Fortschreibung der Entwurfsgrundlage (Design Basis)	Prüfbericht und Konformitätsbescheinigung	
	Ausführungsplanung der primären und sekundären Tragstrukturen	2.3	Ausführungsplanung der primären und sekundären Tragstrukturen	Prüfbericht und Konformitätsbescheinigung	

Phase	Ziele und Maßnahmen	Kapitel	Vom Genehmigungsinhaber zu erstellende Unterlagen, die beim BSH einzureichen sind	Vom Prüfbeauftragten zu erstellende Unterlagen, die beim BSH einzureichen sind	Zulassungs-behörde
Konstruktion	Fortschreibung von Grundlagendokumenten in Form von einer weiteren Konkretisierung der Konzepte: • Festlegung von Brandabschnitten für die Betriebsstruktur (Topside) • Beschreibung aktiver und passiver Brandbekämpfungseinrichtungen • Festlegung der Transport- und Zugangssysteme bei der Trag- und Betriebsstruktur • Fortschreibung des Betriebskonzepts und der Auslegung der Offshore-Station • Fertigstellung des Brandschutzkonzepts für die Offshore-Station • Fertigstellung des Ausrüstungskonzepts für die Offshore-Station • Umsetzung der konstruktiven Anforderungen der Offshore-Station auf Grundlage der Sicherheits- und Funktionsanforderungen aus der Gefährdungsbeurteilung • Fertigstellung des endgültigen Transport- und Zugangskonzepts	5.3		Prüfbericht (auf Basis der Grundlagendokumente)	Plausibilitäts-prüfung 3. Freigabe (ggf. mit Maßgaben)
	Ausführungsplanung für die Errichtung	2.3.4	Errichtungshandbuch	Prüfbericht	
	Fertigstellung der Ausführungsplanung für die Errichtung der Offshore-Station, u. a.: • Erstellung eines Inbetriebnahmekonzepts • Vorlage von Berichten zu durchgeführten Tests • detaillierte Beschreibung von Transport- und Errichtungsvorgängen Erstellung des Umsetzungskonzepts • Erstellung der Schleppvorgänge • Transport- und Installationsvorgänge	2.3.5		Prüfbericht	
	Rückbaukonzept	1.3.3	Rückbaukonzept	Prüfbericht und Konformitäts-bescheinigung	

Phase	Ziele und Maßnahmen	Kapitel	Vom Genehmigungsinhaber zu erstellende Unterlagen, die beim BSH einzureichen sind	Vom Prüfbeauftragten zu erstellende Unterlagen, die beim BSH einzureichen sind	Zulassungsbehörde
Ausführung	Fertigungs- und Montageüberwachung Überprüfung der QM-Zertifikate der Hersteller Erstellung von ZFP-Berichten u. a. Überwachung von Transport, Errichtung und Installation und Inbetriebnahme	2.4		Inspektionsberichte und Konformitätsbescheinigung über Fertigungs- und Montageüberwachung, Transportüberwachung, Installationsüberwachung und Inbetriebnahmeüberwachung	Plausibilitätsprüfung Betriebsfreigabe (ggf. mit Maßgaben)
	Dokumentation des Baubestands	2.4.2.3	Baubestandsplan	Prüfbericht und Konformitätsbescheinigung	
	Betriebsüberwachung	1.3.5 2.4.3 2.5.3	Prüf- und Inspektionsplan für WKP	Prüfbericht und Konformitätsbescheinigung	
		2.4.3	Betriebshandbuch	Prüfbericht und Konformitätsbescheinigung Projektzertifikat	
Betrieb	Wiederkehrende Prüfungen	1.3.5 2.5 3.5 5.5		Prüfbericht und Konformitätsbescheinigung	Plausibilitätsprüfung; Aufrechterhaltung der Betriebserlaubnis
Rückbau	Rückbauplanung	2.6 3.6	Detaillierte Beschreibung des Rückbauvorgangs Entsorgungsnachweise	Prüfbericht und Konformitätsbescheinigung	Plausibilitätsprüfung; Erklärung über ordnungsgemäßen Rückbau und Entsorgung
	Überwachung von Außerbetriebnahme, Rückbau der Komponenten, Transport und Entsorgung	2.6		Inspektionsberichte und Konformitätsbescheinigung	

Tabelle 1-2: Zeitlicher Ablauf für Offshore-Stationen

Appendix 02

Member utilization calculations acc. to ISO 19902

[2] (*Petroleum and Natural Gas Industry - Fixed steel offshore structure,* ISO 19902:2007, International Organization for Standardization, ISO), Pages 97-111.

13 Strength of tubular members

13.1 General

The requirements given in this clause apply to unstiffened and ring stiffened cylindrical tubulars having a thickness $t \geqslant 6$ mm, a diameter to thickness ratio $D/t \leqslant 120$ and material meeting the general requirements of Clause 19. In addition, yield strengths shall be less than 500 MPa and the ratio of yield strength as used to ultimate tensile strength shall not exceed 0,90.

The requirements for the different components and types and combinations of actions are contained in different subclauses, as detailed in Table 13.1-1. Where no reference is included for a particular component and actions, there exists insufficient test data to enable comprehensive design equations to be prepared, and these circumstances shall be assessed on a case-by-case basis.

Table 13.1-1 — Arrangement of requirements for members

Subclause	Component	Actions				
		Tension	Compression	Bending	Shear	Hydrostatic
13.2.2	Tubular	X				
13.2.3	Tubular		X			
13.2.4	Tubular			X		
13.2.5	Tubular				X	
13.2.6	Tubular					X
13.3.2	Tubular	X		X		
13.3.3	Tubular		X	X		
13.4.2	Tubular	X		X		X
13.4.3	Tubular		X	X		X
13.6.3	Cone	X	X	X		
13.6.4	Cone	X	X	X		X
13.7.2.2	Dented tubular	X				
13.7.2.3	Dented tubular		X			
13.7.2.4	Dented tubular			X		
13.7.2.5	Dented tubular				X	
13.7.3.1	Dented tubular	X		X		
13.7.3.2	Dented tubular		X	X		
13.8	Corroded tubular	X	X	X	X	X
13.9.2.2	Grouted tubular	X				
13.9.2.3	Grouted tubular		X			
13.9.2.4	Grouted tubular			X		
13.9.3.1	Grouted tubular	X		X		
13.9.3.2	Grouted tubular		X	X		

For tubulars subjected to hydrostatic pressure, it can be necessary to ensure that the circumferential value of yield strength is consistent with the value adopted in design.

The requirements in this clause assume that the tubular is constructed in accordance with the fabrication tolerances given in Clause 21. The requirements allow structural design to proceed on the basis that stresses due to the forces from the capped-end actions of hydrostatic pressure are either included in or excluded from the analysis.

In Clause 13, y and z are the axes of a tubular cross-section used to define in-plane and out-of-plane behaviour respectively. “In-plane” is the plane common to the longitudinal axis of the brace member under consideration and the longitudinal axes of the chord member providing restraint, while “out-of-plane” is perpendicular to this plane.

In Equations (13.2-1) to (13.9-39), the stresses are always the absolute values of the stresses as computed, and hence — whether tensile or compressive — are always positive.

13.2 Tubular members subjected to tension, compression, bending, shear or hydrostatic pressure

13.2.1 General

Tubular members subjected independently to axial tension, axial compression, bending, shear, or hydrostatic pressure shall be designed to satisfy the strength and stability requirements specified in 13.2.2 to 13.2.6.

13.2.2 Axial tension

Tubular members subjected to axial tensile forces shall be designed to satisfy the following condition:

$$\sigma_t \leqslant \frac{f_t}{\gamma_{R,t}} \tag{13.2-1}$$

where

σ_t is the axial tensile stress due to forces from factored actions;

f_t is the representative axial tensile strength, $f_t = f_y$;

f_y is the representative yield strength, in stress units;

$\gamma_{R,t}$ is the partial resistance factor for axial tensile strength, $\gamma_{R,t}$ = 1,05.

The utilization of a member, U_m, under axial tension shall be calculated from Equation (13.2-2):

$$U_m = \frac{\sigma_t}{f_t / \gamma_{R,t}} \tag{13.2-2}$$

13.2.3 Axial compression

13.2.3.1 General

Tubular members subjected to axial compressive forces shall be designed to satisfy the following condition:

$$\sigma_c \leqslant \frac{f_c}{\gamma_{R,c}} \tag{13.2-3}$$

where

σ_c is the axial compressive stress due to forces from factored actions;

f_c is the representative axial compressive strength, in stress units, see 13.2.3.2;

$\gamma_{R,c}$ is the partial resistance factor for axial compressive strength, $\gamma_{R,c}$ = 1,18.

The utilization of a member, U_m, under axial compression shall be calculated from Equation (13.2-4):

$$U_m = \frac{\sigma_c}{f_c / \gamma_{R,c}} \qquad (13.2\text{-}4)$$

13.2.3.2 Column buckling

In the absence of hydrostatic pressure, the representative axial compressive strength in 13.2.3.1 for tubular members shall be the smaller of the in-plane and the out-of-plane buckling strengths determined from the following equations:

$$f_c = \left(1{,}0 - 0{,}278\lambda^2\right) f_{yc} \quad \text{for } \lambda \leqslant 1{,}34 \qquad (13.2\text{-}5)$$

$$f_c = \frac{0{,}9}{\lambda^2} f_{yc} \quad \text{for } \lambda > 1{,}34 \qquad (13.2\text{-}6)$$

$$\lambda = \sqrt{\frac{f_{yc}}{f_e}} = \frac{K\,L}{\pi\,r}\sqrt{\frac{f_{yc}}{E}} \qquad (13.2\text{-}7)$$

where

- f_c is the representative axial compressive strength, in stress units;
- f_{yc} is the representative local buckling strength, in stress units, see 13.2.3.3;
- λ is the column slenderness parameter;
- f_e is the smaller of the Euler buckling strengths in the y- and z-directions, in stress units, see 13.3.3;
- E is Young's modulus of elasticity;
- K is the effective length factor in the y- or z-direction selected so that $K\,L$ is the larger of the values in the y- and z-directions, see 13.5;
- L is the unbraced length in y- or z-direction;
- r is the radius of gyration, $r = \sqrt{I / A}$;
- I is the moment of inertia of the cross-section;
- A is the cross-sectional area.

13.2.3.3 Local buckling

The representative local buckling strength, f_{yc}, in 13.2.3.2 shall be determined from:

$$f_{yc} = f_y \quad \text{for } \frac{f_y}{f_{xe}} \leqslant 0{,}170 \qquad (13.2\text{-}8)$$

$$f_{yc} = \left(1{,}047 - 0{,}274\frac{f_y}{f_{xe}}\right) f_y \quad \text{for } 0{,}170 < \frac{f_y}{f_{xe}} \qquad (13.2\text{-}9)$$

and

$$f_{xe} = 2\,C_x E\,t / D \qquad (13.2\text{-}10)$$

where

f_y is the representative yield strength, in stress units;

f_{xe} is the representative elastic local buckling strength, in stress units;

C_x is the elastic critical buckling coefficient, see below;

E is Young's modulus of elasticity;

D is the outside diameter of the member;

t is the wall thickness of the member.

The theoretical value of C_x for an ideal tubular is 0,6. However, a reduced value of $C_x = 0{,}3$ should be used in Equation (13.2-10) to account for the effect of initial geometric imperfections within the tolerance limits given in Clause 21. A reduced value of $C_x = 0{,}3$ is implicit in the value of f_{xe} used in Equations (13.2-8) and (13.2-9).

13.2.4 Bending

Tubular members subjected to bending moments shall be designed to satisfy the following condition:

$$\sigma_b = \frac{M}{Z_e} \leqslant \frac{f_b}{\gamma_{R,b}} \tag{13.2-11}$$

where

σ_b is the bending stress due to forces from factored actions; when $M > M_y$, σ_b is to be considered as an equivalent elastic bending stress, $\sigma_b = M/Z_e$;

f_b is the representative bending strength, in stress units, see Equations (13.2-13) to (13.2-15);

$\gamma_{R,b}$ is the partial resistance factor for bending strength, $\gamma_{R,b} = 1{,}05$;

M is the bending moment due to factored actions;

M_y is the elastic yield moment;

Z_e is the elastic section modulus, $Z_e = \frac{\pi}{64}\left(D^4 - (D-2t)^4\right) \Big/ \left(\frac{D}{2}\right)$.

The utilization of a member, U_m, under bending moments shall be calculated from Equation (13.2-12):

$$U_m = \frac{\sigma_b}{f_b / \gamma_{R,b}} = \frac{M / Z_e}{f_b / \gamma_{R,b}} \tag{13.2-12}$$

The representative bending strength for tubular members shall be determined from:

$$f_b = \left(\frac{Z_p}{Z_e}\right) f_y \qquad \text{for } \frac{f_y D}{E t} \leqslant 0{,}0517 \tag{13.2-13}$$

$$f_b = \left[1{,}13 - 2{,}58\left(\frac{f_y D}{E t}\right)\right]\left(\frac{Z_p}{Z_e}\right) f_y \qquad \text{for } 0{,}0517 < \frac{f_y D}{E t} \leqslant 0{,}1034 \tag{13.2-14}$$

$$f_b = \left[0{,}94 - 0{,}76\left(\frac{f_y D}{E t}\right)\right]\left(\frac{Z_p}{Z_e}\right) f_y \qquad \text{for } 0{,}1034 < \frac{f_y D}{E t} \leqslant 120\frac{f_y}{E} \tag{13.2-15}$$

where, additionally,

f_y is the representative yield strength, in stress units;

D is the outside diameter of the member;

t is the wall thickness of the member;

Z_p is the plastic section modulus, $Z_p = \frac{1}{6}\left[D^3 - (D-2t)^3\right]$.

13.2.5 Shear

13.2.5.1 Beam shear

Tubular members subjected to beam shear forces shall be designed to satisfy the following condition:

$$\tau_b = \frac{2V}{A} \leqslant \frac{f_v}{\gamma_{R,v}} \tag{13.2-16}$$

where

τ_b is the maximum beam shear stress due to forces from factored actions;

f_v is the representative shear strength, in stress units, $f_v = f_y / \sqrt{3}$;

$\gamma_{R,v}$ is the partial resistance factor for shear strength, $\gamma_{R,v} = 1{,}05$;

V is the beam shear due to factored actions, in force units;

A is the cross-sectional area.

The utilization of a member, U_m, under beam shear shall be calculated from Equation (13.2-17):

$$U_m = \frac{\tau_b}{f_v / \gamma_{R,v}} = \frac{2V / A}{f_v / \gamma_{R,v}} \tag{13.2-17}$$

13.2.5.2 Torsional shear

Tubular members subjected to torsional shear forces shall be designed to satisfy the following condition:

$$\tau_t = \frac{M_{v,t} D}{2 I_p} \leqslant \frac{f_v}{\gamma_{R,v}} \tag{13.2-18}$$

where, in addition to the definitions in 13.2.5.1,

τ_t is the torsional shear stress due to forces from factored actions;

$M_{v,t}$ is the torsional moment due to factored actions;

I_p is the polar moment of inertia, $I_p = \frac{\pi}{32}\left[D^4 - \left(D - 2t\right)^4\right]$.

The partial resistance factor, $\gamma_{R,v}$, for shear, is the same for both torsional shear and beam shear, see 13.2.5.1.

The utilization of a member, U_m, under torsional shear shall be calculated from Equation (13.2-19):

$$U_m = \frac{\tau_t}{f_v / \gamma_{R,v}} = \frac{M_{v,t} D / 2 I_p}{f_v / \gamma_{R,v}} \tag{13.2-19}$$

13.2.6 Hydrostatic pressure

13.2.6.1 Calculation of hydrostatic pressure

The effective depth at the location being checked shall be calculated taking into account the depth of the member below still water level and the effect of passing waves. The factored hydrostatic pressure (p) shall be calculated from Equation (13.2-20):

$$p = \gamma_{f,G1}\, \rho_w\, g\, H_z \tag{13.2-20}$$

where

$\gamma_{f,G1}$ is the partial action factor for permanent actions 1, see Table 9.10-1;

ρ_w is the density of the sea water which may be taken as 1 025 kg/m^3;

g is the acceleration due to gravity (m/s^2);

H_z is the effective hydrostatic head (m)

$$H_z = -z + \frac{H_w}{2} \frac{\cosh\left[k\left(d+z\right)\right]}{\cosh\left(k\, d\right)} \tag{13.2-21}$$

where

z is the depth of the member relative to still water level (measured positive upwards);

d is the still water depth to the sea floor;

H is the wave height;

k is the wave number, $k = 2\,\pi/\lambda$;

where

λ is the wave length.

For installation conditions, z shall be the maximum depth of submergence during launch, or the maximum differential head during the upending and installation sequence plus an amount to allow for deviations from the planned sequence, and $\gamma_{f,G1}$ in Equation (13.2-20) shall be replaced by $\gamma_{f,T}$, see Clause 8.

13.2.6.2 Hoop buckling

Tubular members subjected to external pressure shall be designed to satisfy the following condition:

$$\sigma_h = \frac{p\,D}{2\,t} \leqslant \frac{f_h}{\gamma_{R,h}} \tag{13.2-22}$$

where

σ_h is the hoop stress due to the forces from factored hydrostatic pressure;

p is the factored hydrostatic pressure, see 13.2.6.1;

D is the outside diameter of the member;

t is the wall thickness of the member;

f_h is the representative hoop buckling strength, in stress units, see Equations (13.2-23) to (13.2-25);

$\gamma_{R,h}$ is the partial resistance factor for hoop buckling strength, $\gamma_{R,h}$ = 1,25.

For tubular members satisfying the out-of-roundness tolerances given in Annex G, f_h shall be determined from:

$$f_h = f_y \qquad \text{for } f_{he} > 2{,}44\, f_y \tag{13.2-23}$$

$$f_h = 0{,}7\left(f_{he}/f_y\right)^{0{,}4} f_y \leqslant f_y \qquad \text{for } 0{,}55\, f_y < f_{he} \leqslant 2{,}44\, f_y \tag{13.2-24}$$

$$f_h = f_{he} \qquad \text{for } f_{he} \leqslant 0{,}55\, f_y \tag{13.2-25}$$

where

f_y is the representative yield strength, in stress units;

f_{he} is the representative elastic critical hoop buckling strength, in stress units

$$f_{he} = 2C_h E t / D \tag{13.2-26}$$

where the elastic critical hoop buckling coefficient C_h is:

$$C_h = 0{,}44\, t/D \qquad \text{for } \mu \geqslant 1{,}6\, D/t \tag{13.2-27}$$

$$C_h = 0{,}44\, t/D + 0{,}21\,(D/t)^3\,\mu^4 \qquad \text{for } 0{,}825\, D/t \leqslant \mu < 1{,}6\, D/t \tag{13.2-28}$$

$$C_h = 0{,}737/(\mu - 0{,}579) \qquad \text{for } 1{,}5 \leqslant \mu < 0{,}825\, D/t \tag{13.2-29}$$

$$C_h = 0{,}80 \qquad \text{for } \mu < 1{,}5 \tag{13.2-30}$$

where μ is a geometric parameter and

$$\mu = \frac{L_r}{D}\sqrt{\frac{2D}{t}}$$

where L_r is the length of tubular between stiffening rings, diaphragms, or end connections.

For tubular members exceeding the out-of-roundness tolerances, see A.13.2.6.2.

The utilization of a member, U_m, under external pressure shall be calculated from Equation (13.2-31):

$$U_m = \frac{\sigma_h}{f_h / \gamma_{R,h}} = \frac{p\,D / 2\,t}{f_h / \gamma_{R,h}} \tag{13.2-31}$$

13.2.6.3 Ring stiffener design

For $\mu \geqslant 1{,}6\,D/t$, the elastic critical hoop buckling stress is approximately equal to that of a long unstiffened tubular. Hence, to be effective, stiffening rings, if required, should be spaced such that

$$L_r < 1{,}6\sqrt{\frac{D^3}{2\,t}} \tag{13.2-32}$$

The circumferential stiffening ring size may be calculated from Equations (13.2-33) or (13.2-34) as appropriate, provided, if the yield strength of the ring stiffener is less than that of the member, that this smaller value of yield strength is used instead of f_y in Equations (13.2-23) to (13.2-25).

$$I_c = f_{he} \frac{t\,L_r\,D^2}{8\,E} \quad \text{for internal rings} \tag{13.2-33}$$

$$I_c = f_{he} \frac{t\,L_r\,D_r^2}{8\,E} \quad \text{for external rings} \tag{13.2-34}$$

Where, in addition to the definitions given in 13.2.6.2,

- I_c is the required moment of inertia for the composite ring section;
- L_r is the ring spacing;
- D is the outside diameter of the member;
- D_r is the diameter of the centroid of the composite ring section;
- E is Young's modulus of elasticity.

The composite ring section may be assumed to include an effective width of the member wall of $1{,}1\sqrt{D\,t}$.

Where out-of-roundness is in excess of that permitted by Annex G, larger stiffeners can be required. In such cases the bending due to excess out-of-roundness shall be specifically investigated.

Local buckling of ring stiffeners with flanges may be excluded as a possible failure mode, provided that the following requirements are fulfilled:

$$\frac{h}{t_w} \leqslant 1{,}1\sqrt{\frac{E}{f_{y,r}}} \tag{13.2-35}$$

and

$$\frac{b}{t_f} \leqslant 0{,}6\sqrt{\frac{E}{f_{y,r}}} \tag{13.2-36}$$

where, in addition,

h is the web height;

t_w is the web thickness;

b is half the flange width of T stiffeners or the full flange width for angle stiffeners;

t_f is the flange thickness;

$f_{y,r}$ is the representative yield strength of the ring stiffeners, in stress units.

Local buckling of ring stiffeners without flanges may be excluded as a possible failure mode, provided that

$$\frac{h}{t_w} \leqslant 0{,}6\sqrt{\frac{E}{f_{y,r}}} \qquad (13.2\text{-}37)$$

Ring stiffeners, including their components and whether internal or external, shall have a minimum thickness of 10 mm.

13.3 Tubular members subjected to combined forces without hydrostatic pressure

13.3.1 General

The following gives requirements for members subjected to combined forces, which give rise to global and local interactions between axial forces and bending moments, without hydrostatic pressure. Generally, the secondary moments from factored global actions and the associated bending stresses (P–Δ effects) do not need to be considered. However, when the axial member force is substantial, or when the component on which the axial force acts is very flexible, the secondary moments due to P–Δ effects from factored global actions should be taken into account.

13.3.2 Axial tension and bending

Tubular members subjected to combined axial tension and bending forces shall be designed to satisfy the following condition at all cross-sections along their length:

$$\frac{\gamma_{R,t}\,\sigma_t}{f_t} + \frac{\gamma_{R,b}\sqrt{\sigma_{b,y}^2 + \sigma_{b,z}^2}}{f_b} \leqslant 1{,}0 \qquad (13.3\text{-}1)$$

where, in addition to the definitions in 13.2.2 and 13.2.4

$\sigma_{b,y}$ is the bending stress about the member y-axis (in-plane) due to forces from factored actions;

$\sigma_{b,z}$ is the bending stress about the member z-axis (out-of-plane) due to forces from factored actions.

The utilization of a member, U_m, under combined axial tension and bending shall be calculated from Equation (13.3-2):

$$U_m = \frac{\gamma_{R,t}\,\sigma_t}{f_t} + \frac{\gamma_{R,b}\sqrt{\sigma_{b,y}^2 + \sigma_{b,z}^2}}{f_b} \qquad (13.3\text{-}2)$$

13.3.3 Axial compression and bending

Tubular members subjected to combined axial compression and bending forces shall be designed to satisfy the following conditions at all cross-sections along their length:

$$\frac{\gamma_{R,c}\ \sigma_c}{f_c} + \frac{\gamma_{R,b}}{f_b}\left[\left(\frac{C_{m,y}\ \sigma_{b,y}}{1-\sigma_c / f_{e,y}}\right)^2 + \left(\frac{C_{m,z}\ \sigma_{b,z}}{1-\sigma_c / f_{e,z}}\right)^2\right]^{0,5} \leqslant 1,0 \qquad (13.3\text{-}3)$$

and

$$\frac{\gamma_{R,c}\ \sigma_c}{f_{yc}} + \frac{\gamma_{R,b}\sqrt{\sigma_{b,y}^2 + \sigma_{b,z}^2}}{f_b} \leqslant 1,0 \qquad (13.3\text{-}4)$$

where, in addition to the definitions given in 13.2.3, 13.2.4 and 13.3.2,

$C_{m,y}$, $C_{m,z}$ are the moment reduction factors corresponding to the member y- and z-axes, respectively (see 13.5);

$f_{e,y}$, $f_{e,z}$ are the Euler buckling strengths corresponding to the member y- and z-axes, respectively, in stress units

$$f_{e,y} = \frac{\pi^2 E}{\left(K_y\ L_y / r\right)^2} \qquad (13.3\text{-}5)$$

$$f_{e,z} = \frac{\pi^2 E}{\left(K_z\ L_z / r\right)^2} \qquad (13.3\text{-}6)$$

where

K_y, K_z are the effective length factors for the y- and z-directions, respectively, see 13.5;

L_y, L_z are the unbraced lengths in the y- and z-directions, respectively.

The utilization of a member, U_m, under axial compression and bending shall be the larger value calculated from Equations (13.3-7) and (13.3-8):

$$U_m = \frac{\gamma_{R,c}\ \sigma_c}{f_c} + \frac{\gamma_{R,b}}{f_b}\left[\left(\frac{C_{m,y}\ \sigma_{b,y}}{1-\sigma_c / f_{e,y}}\right)^2 + \left(\frac{C_{m,z}\ \sigma_{b,z}}{1-\sigma_c / f_{e,z}}\right)^2\right]^{0,5} \qquad (13.3\text{-}7)$$

$$U_m = \frac{\gamma_{R,c}\ \sigma_c}{f_{yc}} + \frac{\gamma_{R,b}\sqrt{\sigma_{b,y}^2 + \sigma_{b,z}^2}}{f_b} \qquad (13.3\text{-}8)$$

13.3.4 Piles

Overall column buckling is normally not a problem in the design of pile segments below the sea floor because the surrounding soils inhibit overall column buckling. However, whenever laterally loaded piles are subjected to significant axial actions, the secondary moments (P–Δ effects) should be considered in stress computations. An effective method of analysis is to model the pile as a beam-column on an elastic foundation. When such an analysis is used, the pile segment should be designed to satisfy Equation (13.3-4), except that $\sigma_{b,y}$ and $\sigma_{b,z}$ in this formula should include the stresses from the secondary moments (P–Δ effects) computed from factored actions.

13.4 Tubular members subjected to combined forces with hydrostatic pressure

13.4.1 General

A tubular member below the water line is subjected to hydrostatic pressure unless it has been flooded. Flooding is normally only used for a structure's legs in order to assist in upending and placement and for pile installation. Even where members are flooded in the in-place condition, they can be subjected to hydrostatic pressure during launch and installation. The effects of hydrostatic pressure shall be taken into account when conducting member checks, including the axial components of such pressure (i.e. capped-end actions). When conducting an analysis of the axial components of hydrostatic pressure, such action effects can be taken directly into account during the analysis or can be included subsequently. The formulations presented in 13.4 allow either approach for accounting for the axial effects of hydrostatic pressure to be used.

When checking tubular members subjected to hydrostatic pressure, four checks are required:

a) check for hoop buckling under hydrostatic pressure alone, Equation (13.2-22);

b) check for tensile yielding when the combination of action effects, including those due to capped-end forces, results in tension in the member, 13.4.2;

c) check for compression yielding and local buckling when the combination of action effects, including those due to capped-end forces, results in compression in the member, 13.4.3;

d) check for column buckling when the action effects, excluding those due to capped-end forces, result in compression in the member, 13.4.3.

For analyses using factored actions that include capped-end actions:

$\sigma_{t,c}$ is the axial tensile stress due to forces from factored actions;

$\sigma_{c,c}$ is the axial compressive stress due to forces from factored actions.

For analyses using factored actions that do not include the capped-end actions:

$$\sigma_{t,c} = \sigma_t - \sigma_q \qquad \text{if } \sigma_t \geqslant \sigma_q \tag{13.4-1}$$

$$\sigma_{c,c} = \sigma_q - \sigma_t \qquad \text{if } \sigma_t < \sigma_q \tag{13.4-2}$$

$$\sigma_{c,c} = \sigma_c + \sigma_q \tag{13.4-3}$$

where

σ_t is the axial tensile stress due to forces from factored actions without capped-end actions;

σ_c is the axial compressive stress due to forces from factored actions without capped-end actions;

σ_q is the compressive axial stress due to the capped-end hydrostatic actions calculated using the value of pressure from Equation (13.2-20).

NOTE 1 In some circumstances, the use of factored actions leads to conditions in which $\sigma_{t,c}$ is tensile, whereas under unfactored actions $\sigma_{t,c}$ is compressive. These cases usually occur for relatively low values of $\sigma_{t,c}$ and the error is not considered to be significant.

The capped-end stresses (σ_q) may be approximated as half the hoop stress due to forces from factored hydrostatic pressure, i.e.

$$\sigma_q = 0{,}5\ \sigma_h \tag{13.4-4}$$

NOTE 2 In accordance with 13.1, σ_q always has a positive value.

In reality, the magnitude of these stresses depends on the restraint on the member provided by the rest of the structure and its value can be more or less than 0,5 σ_h. The approximation 0,5 σ_h may be replaced by a stress computed from a more rigorous analysis, using factored actions.

When an analysis uses factored actions that include capped-end actions, σ_c for net compression cases may be approximated by

$$\sigma_c = \sigma_q - \sigma_{t,c} \qquad \text{if } \sigma_{t,c} < \sigma_q \tag{13.4-5}$$

$$\sigma_c = \sigma_{c,c} - \sigma_q \qquad \text{if } \sigma_{c,c} > \sigma_q \tag{13.4-6}$$

13.4.2 Axial tension, bending and hydrostatic pressure

Tubular members subjected to combined axial tension, bending and hydrostatic pressure shall be designed to satisfy the following requirements at all cross-sections along their length.

$$\frac{\gamma_{R,t}\sigma_{t,c}}{f_{t,h}} + \frac{\gamma_{R,b}\sqrt{\sigma_{b,y}^2 + \sigma_{b,z}^2}}{f_{b,h}} \leqslant 1{,}0 \tag{13.4-7}$$

where, in addition to the definitions in 13.2, 13.3 and 13.4.1:

$f_{t,h}$ is the representative axial tensile strength in the presence of external hydrostatic pressure, in stress units

$$f_{t,h} = f_y\left(\sqrt{1+0{,}09B^2 - B^{2\eta}} - 0{,}3B\right) \tag{13.4-8}$$

$f_{b,h}$ is the representative bending strength in the presence of external hydrostatic pressure, in stress units

$$f_{b,h} = f_b\left(\sqrt{1+0{,}09B^2 - B^{2\eta}} - 0{,}3B\right) \tag{13.4-9}$$

and

$$B = \frac{\gamma_{R,h}\sigma_h}{f_h}, \qquad B \leqslant 1{,}0 \tag{13.4-10}$$

$$\eta = 5 - 4\frac{f_h}{f_y} \tag{13.4-11}$$

The utilization of a member, U_m, under axial tension, bending and hydrostatic pressure shall be calculated from Equation (13.4-12):

$$U_m = \frac{\gamma_{R,t}\ \sigma_{t,c}}{f_{t,h}} + \frac{\gamma_{R,b}\sqrt{\sigma_{b,y}^2 + \sigma_{b,z}^2}}{f_{b,h}} \tag{13.4-12}$$

13.4.3 Axial compression, bending and hydrostatic pressure

Tubular members subjected to combined axial compression, bending and hydrostatic pressure shall be designed to satisfy the following requirements at all cross-sections along their length.

$$\frac{\gamma_{R,c}\,\sigma_{c,c}}{f_{yc}} + \frac{\gamma_{R,b}\sqrt{\sigma_{b,y}^2 + \sigma_{b,z}^2}}{f_{b,h}} \leqslant 1{,}0 \tag{13.4-13}$$

$$\frac{\gamma_{R,c}\,\sigma_c}{f_{c,h}} + \frac{\gamma_{R,b}}{f_{b,h}}\left[\left(\frac{C_{m,y}\,\sigma_{b,y}}{1-\sigma_c/f_{e,y}}\right)^2 + \left(\frac{C_{m,z}\,\sigma_{b,z}}{1-\sigma_c/f_{e,z}}\right)^2\right]^{0,5} \leqslant 1{,}0 \tag{13.4-14}$$

where, additionally:

$f_{c,h}$ is the representative axial compressive strength in the presence of external hydrostatic pressure, in stress units

$$f_{c,h} = \frac{1}{2} f_{yc}\left[\left(1{,}0 - 0{,}278\lambda^2\right) - \frac{2\sigma_q}{f_{yc}} + \sqrt{\left(1{,}0 - 0{,}278\lambda^2\right)^2 + 1{,}12\lambda^2\frac{\sigma_q}{f_{yc}}}\right] \quad \text{for } \lambda \leqslant 1{,}34\sqrt{\left(1 - \frac{2\sigma_q}{f_{yc}}\right)^{-1}} \tag{13.4-15}$$

$$f_{c,h} = \frac{0{,}9}{\lambda^2} f_{yc} \quad \text{for } \lambda > 1{,}34\sqrt{\left(1 - \frac{2\sigma_q}{f_{yc}}\right)^{-1}} \tag{13.4-16}$$

If the maximum combined compressive stress, $\sigma_x = \sigma_b + \sigma_{c,c}$, and the representative elastic local buckling strength, f_{xe}, exceed the limits given below, then Equation (13.4-18) shall also be satisfied:

$$\sigma_x > 0{,}5\frac{f_{he}}{\gamma_{R,h}} \quad \text{and} \quad \frac{f_{xe}}{\gamma_{R,c}} > 0{,}5\frac{f_{he}}{\gamma_{R,h}} \tag{13.4-17}$$

$$\frac{\sigma_x - 0{,}5 f_{he}/\gamma_{R,h}}{f_{xe}/\gamma_{R,c} - 0{,}5 f_{he}/\gamma_{R,h}} + \left(\frac{\gamma_{R,h}\sigma_h}{f_{he}}\right)^2 \leqslant 1{,}0 \tag{13.4-18}$$

where

f_{he} is the representative elastic critical hoop buckling strength defined in 13.2.6.2;

f_{xe} is the representative elastic local buckling strength defined in 13.2.3.3.

The utilization of a member, U_m, under axial compression, bending and hydrostatic pressure shall be the largest value calculated from Equations (13.4-19), (13.4-20) and (13.4-21):

$$U_m = \frac{\gamma_{R,c}\,\sigma_{c,c}}{f_{yc}} + \frac{\gamma_{R,b}\sqrt{\sigma_{b,y}^2 + \sigma_{b,z}^2}}{f_{b,h}} \quad \text{when Equation (13.4-13) applies} \tag{13.4-19}$$

$$U_m = \frac{\gamma_{R,c}\,\sigma_c}{f_{c,h}} + \frac{\gamma_{R,b}}{f_{b,h}}\left[\left(\frac{C_{m,y}\,\sigma_{b,y}}{1-\sigma_c/f_{e,y}}\right)^2 + \left(\frac{C_{m,z}\,\sigma_{b,z}}{1-\sigma_c/f_{e,z}}\right)^2\right]^{0,5} \quad \text{when Equation (13.4-14) applies} \tag{13.4-20}$$

$$U_m = \frac{\sigma_x - 0{,}5 f_{he}/\gamma_{R,h}}{f_{xe}/\gamma_{R,c} - 0{,}5 f_{he}/\gamma_{R,h}} + \left(\frac{\gamma_{R,h}\,\sigma_h}{f_{he}}\right)^2 \quad \text{when Equation (13.4-18) applies} \tag{13.4-21}$$

13.5 Effective lengths and moment reduction factors

The effective lengths and moment reduction factors may be determined using a rational analysis that includes joint flexibility and side-sway. In lieu of such a rational analysis, values of effective length factors (K) and moment reduction factors (C_m) may be taken from Table 13.5-1. These factors

a) do not apply to cantilever members, and

b) assume both member ends are rotationally restrained in both planes of bending (see A.13.5).

NOTE Examples of the use of rational analysis can be found in the relevant references cited in Annex A.

Lengths to which the effective length factors K are applied are normally measured from centreline to centreline of the end joints. However, for members framing into legs, the following modified lengths may be used, provided that no interaction between the buckling of members and legs affects the utilization of the legs:

— face-of-leg to face-of-leg for main diagonal braces;

— face-of-leg to centreline of end joint for K-braces.

Lower K factors than those according to Table 13.5-1 may be used provided they are supported by more rigorous analysis.

Table 13.5-1 — Effective length and moment reduction factors for member strength checking

Structural component	K	C_m [a]
Topsides legs		
Braced	1,0	1)
Portal (unbraced)	K [b]	1)
Structure legs and piling		
Grouted composite section	1,0	3)
Ungrouted legs	1,0	3)
Ungrouted piling between shim points	1,0	2)
Structure brace members		
Primary diagonals and horizontals	0,7	2) or 3)
K- braces [c]	0,7	2) or 3)
X -braces		
Longer segment length [c]	0,8	2) or 3)
Full length [d]	0,7	2) or 3)
Secondary horizontals	0,7	2) or 3)

[a] C_m values for the three cases defined in this table are as follows:

1) 0,85;

2) for members with no transverse loading, other than self weight,

$$C_m = 0,6 - 0,4 \times M_1/M_2$$

where M_1/M_2 is the ratio of smaller to larger moments at the ends of the unbraced portion of the member in the plane of bending under consideration;

M_1/M_2 is positive when the member is bent in reverse curvature, negative when bent in single curvature.

C_m shall not be larger than 0,85;

3) for members with transverse loading, other than self weight,

$C_m = 1,0 - 0,4 \times (\sigma_c/f_e)$, or 0,85, whichever is less,

and $f_e = f_{ey}$ or f_{ez} as appropriate.

[b] See effective length alignment chart in A.13.5. This may be modified to account for conditions different from those assumed in the development of the chart.

[c] For either in-plane or out-of-plane effective lengths, at least one pair of members framing into a K- or X-joint shall be in tension, if the joint is not braced out-of-plane.

[d] When all members are in compression and the joint is not braced out-of-plane.

Appendix 03

Joint classification and utilization calculations acc. to ISO 19902

[2] (*Petroleum and Natural Gas Industry - Fixed steel offshore structure,* ISO 19902:2007, International Organization for Standardization, ISO), Pages 134-149.

$$K_{30} = \left(0{,}04 - \theta/15\right) \geqslant 0 \tag{13.9-33}$$

$$\theta = \frac{0{,}67 A_{\text{g}} \left(f_{\text{cu}} + \dfrac{C_1 f_{\text{y}}\ t}{D} \right)}{f_{\text{ug}}\ A_{\text{tr}}} \tag{13.9-34}$$

β is the ratio of the smaller to the larger end moment, with $\beta = 1$ if no end moments apply;

$$C_1 = 4\phi\varepsilon\left(1+\phi+\phi^2\right)^{-0{,}5} \tag{13.9-35}$$

$$\phi = 0{,}02\left(25-\frac{K\ L}{D}\right) \geqslant 0 \tag{13.9-36}$$

$$\varepsilon = 0{,}25\left(25-\frac{K\ L}{D}\right) \geqslant 0 \tag{13.9-37}$$

The utilization of a grouted tubular member, $U_{\text{m,g}}$, under axial compression and bending shall be the larger value calculated from Equations (13.9-38) and (13.9-39):

$$U_{\text{m,g}} = \frac{\gamma_{\text{R,c,g}}\ \sigma_{\text{c,g}}}{f_{\text{c,g}}} + \frac{\gamma_{\text{R,b,g}}\ T_1\ \sigma_{\text{b,g}}}{f_{\text{b,g}}} + \frac{T_2 \left(\gamma_{\text{R,b,g}}\ \sigma_{\text{b,g}}\right)^2}{f_{\text{b,g}}^{\ 2}} \tag{13.9-38}$$

$$U_{\text{m,g}} = \frac{\gamma_{\text{R,b,g}}\ \sigma_{\text{b,g}}}{f_{\text{b,g}}} \tag{13.9-39}$$

14 Strength of tubular joints

14.1 General

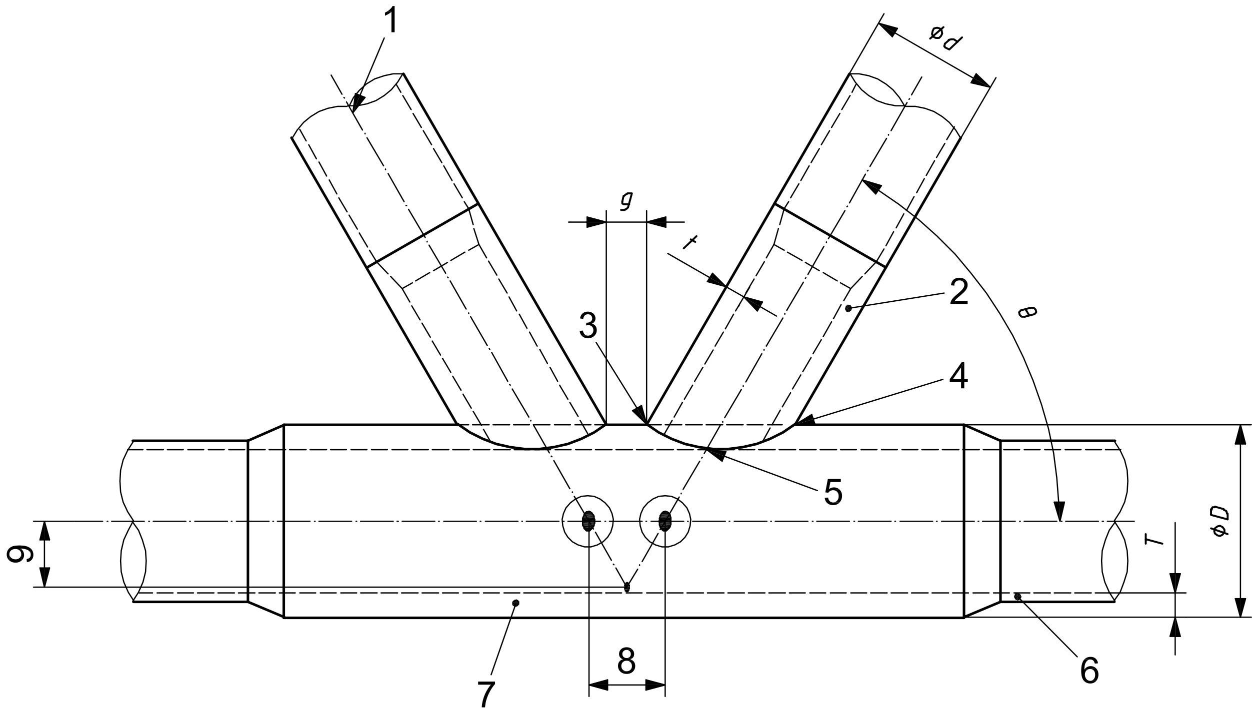
The requirements given in this clause apply to the static design of tubular joints formed by the connection of two or more members. Generic requirements for non-tubular joints are also given. Joint types are classified in 14.2.4.

In lieu of the requirements in this International Standard, reasonable alternative methods may be used for the design of joints. Test data and analytical techniques may be used as a basis for design, provided that it can be demonstrated that the strength of such joints can be determined reliably. Such analytical or numerical techniques should always be calibrated and benchmarked to suitable test data.

The requirements have been derived from a consideration of the representative strength (as opposed to the mean strength) of tubular joints. Representative strength is comparable to lower bound strength. The background to, and a discussion of, the requirements are presented in A.14.

Care should be taken in using the results of very limited test programmes or analytical investigations to provide an estimate of joint strength, since very limited test programmes form an improper basis for determining the representative value (see 7.7). Consideration shall, in such cases, be given to the imposition of a reduction factor on the calculation of joint strength, in order to account for the small amount of data or for a poor basis of the calculations.

The nomenclature for simple joints is given in Figure 14.1-1.



Key

1	brace	θ	included angle between chord and brace axes
2	stub (where present)	g	gap between braces, negative for overlapped stubs
3	crown toe	t	brace wall thickness at intersection
4	crown heel	T	chord wall thickness at intersection
5	saddle	d	brace outside diameter
6	chord	D	chord outside diameter
7	can		
8	offset		
9	eccentricity		

$\beta = \frac{d}{D}$ $\gamma = \frac{D}{2T}$ $\tau = \frac{t}{T}$

Figure 14.1-1 — Terminology and geometrical parameters for simple tubular joints

14.2 Design considerations

14.2.1 Materials

The general requirements for materials are given in Clause 19, while additional requirements specific to the strength of tubular connections are given below.

The representative yield strength of the steel shall be taken as the specified minimum yield strength (SMYS), except that for chord materials with a SMYS of 500 MPa or less, the representative yield strength shall not exceed 80 % of the tensile strength. A.14.2.1 gives additional information on materials with a minimum specified yield strength greater than 500 MPa.

Welds in fabricated joints shall be designed to develop a strength greater than or equal to both the yield strength of the nominal brace cross-section (ignoring any brace stubs) and the full strength of the joint. Further guidelines for welds for circular tubular joints are given in Clause 20.

Joints often involve welds from several brace connections in close proximity. The high restraint of joints can cause large strain concentrations and a potential for cracking or lamellar tearing. Hence the chord material (and brace/stub material, if overlapping is present) shall have adequate through-thickness toughness, see Clause 19.

There is sometimes uncertainty in the material properties in structures that are being assessed (see Clause 24) or reused (see Clause 25). In these instances, testing of material removed from the actual structure can be required. If the through-thickness toughness of joint can steel cannot be determined, inspection for possible cracks or lamellar tearing shall be considered.

Recommendations for grout materials for use in grouted joints are given in 19.6.

14.2.2 Design forces and joint flexibility

Joints shall be designed and assessed using internal forces resulting from factored actions in accordance with Clauses 8 to 11. In addition, for the design of new structures, joints for all primary structural members shall be at least as strong as the adjoining braces, see 14.2.3.

The reduction in secondary (deflection induced) bending moments due to joint flexibility or due to inelastic relaxation may be considered. For ultimate strength analysis of the structure, information on the force-deformation characteristics of joints may be used. These characteristics are dependent on the joint type, configuration, geometry, material properties, load case under consideration and, in certain instances, hydrostatic pressure effects; see A.14.2.2 for a further discussion on joint flexibility.

14.2.3 Minimum strength

The requirement for the strength of joints is given in general form in Equation (14.2-1):

$$S_{\mathrm{j}} \leqslant \frac{R_{\mathrm{j}}}{\gamma_{\mathrm{R,j}}} \tag{14.2-1}$$

where

S_{j} is the generalized internal force in the joint;

R_{j} is the corresponding generalized resistance of the joint;

$\gamma_{\mathrm{R,j}}$ is the partial resistance factor for joints, $\gamma_{\mathrm{R,j}} = 1,05$.

All joints, except those identified as being non-critical, shall additionally be checked to ensure that joint strength exceeds the brace member strength, using Equation (14.2-2):

$$\frac{\gamma_{\mathrm{R,j}}\, S_{\mathrm{j}}}{R_{\mathrm{j}}} \leqslant \frac{U_{\mathrm{b}}}{\gamma_{\mathrm{zj}}} \tag{14.2-2}$$

where, additionally,

U_{b} is the utilization of the brace (see Clause 13) at the end adjoining the joint, which may conservatively be taken as the maximum utilization along the brace or even more conservatively as unity;

γ_{zj} is an extra partial resistance factor to ensure that members fail before the joint yields.

The total resistance factor for joint strength in relation to brace utilization is the product of γ_{R} and γ_{zj}. γ_{zj} shall normally be taken as 1,17, giving a total resistance factor of 1,23; γ_{zj} may be relaxed to a value within the range 1,00 to 1,17 only if this can be justified by the designer, giving a total resistance factor between 1,05 and 1,23.

Non-critical joints are joints that do not

— influence the reserve strength of a structure,

— influence the response of a structure when subjected to accidental events, or

— cause significant safety or environmental consequences if they fail.

In practice, the checks in Equations (14.2-1) and (14.2-2) for combined forces and moments consider the interaction between the forces, the moments, the resistances to forces and the resistances to moments. The strength of a simple tubular joint shall be checked using the interaction equation, see Equation (14.3-12). Interaction for simple tubular joints shall be checked using Equation (14.3-13), except for those joints identified as being non-critical.

14.2.4 Joint classification

There are three basic planar joint types, these being Y-, K- and X-joints, as shown in Figure 14.2-1 and as described below.

— A Y-joint consists of a chord and one brace. Axial force in the brace is reacted by an axial force and beam shear in the chord.

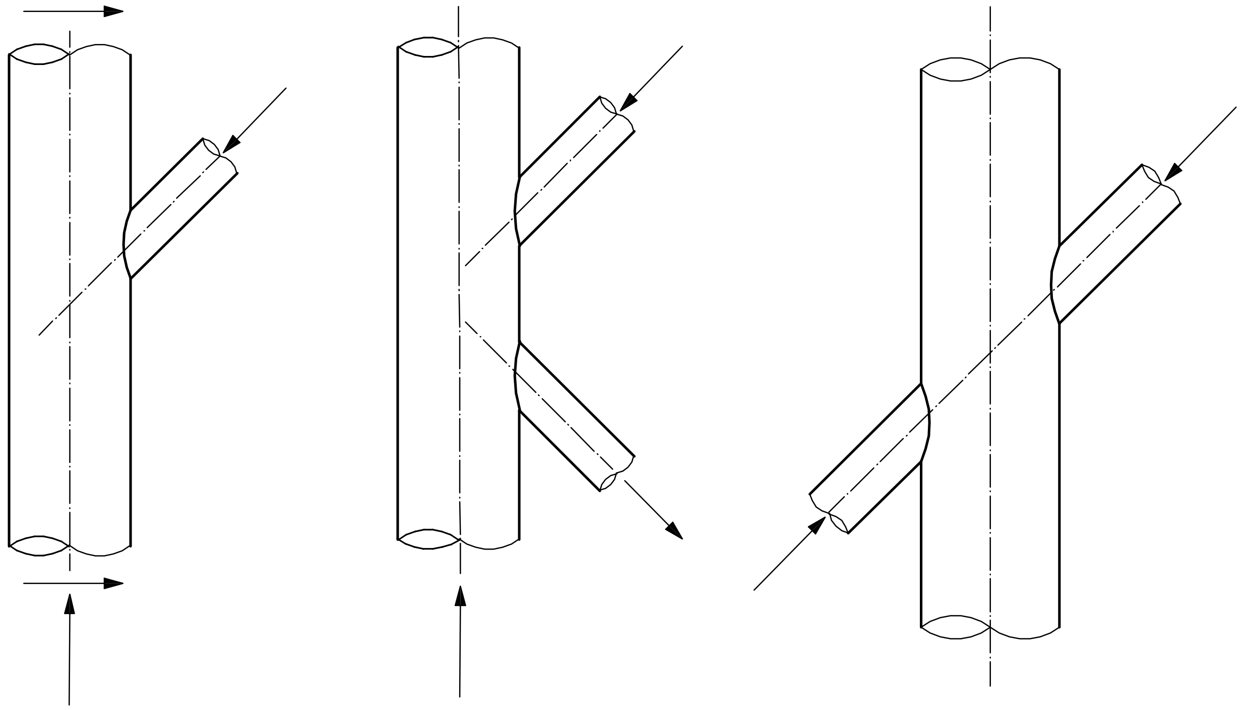
— A K-joint consists of a chord and two braces on the same side of the chord. The components of the axial brace forces normal to the chord balance each other, while the components parallel to the chord add and are reacted by an axial force in the chord.

— An X-joint (also called *cross-joint*) consists of a chord and two braces, one on each side of the chord, where the second brace is a continuation of the first brace. Axial force in one brace is transferred through the chord to the other brace without an overall reaction in the chord.

In all joint types, the chord is the through member.

Many joints are combinations of the above joint types, containing mixtures of behaviour either in one plane or in several planes (multi-planar joints). A T-joint is a Y-joint in which the angle between the brace and the chord is approximately 90°. A DT-joint, or double T-joint, looks like an X-joint with angles of approximately 90° but behaves as two T-joints, in that the axial brace forces are transferred to the chord rather than crossing the chord to the other brace.

Joint classification between Y-, K- and X-joints is based solely on consideration of the axial forces in the braces.



a) Y-joint b) K-joint c) X-joint

Figure 14.2-1 — Basic planar joint types

The design strength of most joints can be determined using the parametric formulae given in 14.3 for the three basic planar joint types. However, fixed steel offshore structures are normally space frames, containing both multiplanar joints and simple Y-, K- and X-joints. The practical use of the basic joint formulae shall reflect, as closely as possible, the force pattern assumed in deriving the formulae by classifying each combination of brace(s) and chord according to the flow of the axial force in the brace(s). A joint should be classified as combinations of Y-, K- and X-joints when the behaviour of the braces contains elements of the behaviour of more than one type. The following approach shall be followed.

Classification as a Y-, K- or X-joint shall apply to the combination of an individual brace with the chord, rather than to the whole joint, on the basis of the axial force pattern for each load case. This classification is relevant to both fatigue and strength considerations.

The classification of each individual brace-chord combination for a given load case shall be as a Y-, K- or X-joint. If the brace-chord combination carries part of the axial brace force as a K-joint, and part as a Y-joint or X-joint, it shall be classified as a proportion of each relevant type, e.g. 50 % as a K-joint and 50 % as an X-joint.

The subdivision in Y-, K- and X-joint axial force patterns normally considers all members in one plane at a joint; brace planes within $\pm 15°$ of each other may be considered as being in the same plane.

The classification should be based on the following:

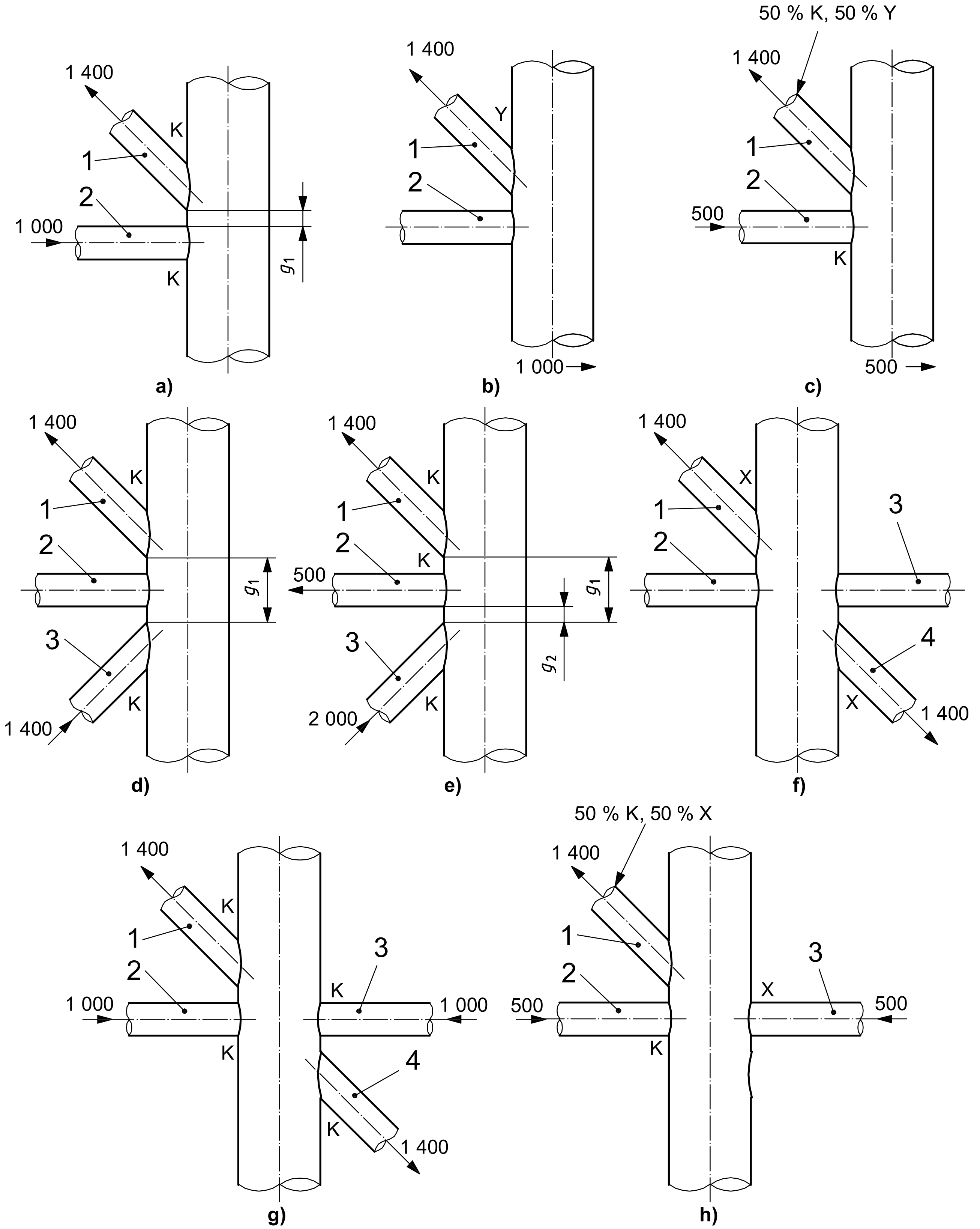
a) a brace should be classified as a K-joint only if the component of axial force in the brace perpendicular to the chord is balanced to within 10 % by force components (perpendicular to the chord) in other braces in the same plane and on the same side of the joint;

b) a brace should be considered as a Y-joint if it does not meet the criteria for a K-joint and if the component of axial force in the brace perpendicular to the chord is reacted as beam shear in the chord;

c) a brace should be considered as an X-joint if it does not meet the criteria for a K-joint or a Y-joint; in this classification the axial force in the brace is transferred through the chord to the opposite side (e.g. to other braces, to padeyes, launch rails or similar structural components).

Figure 14.2-2 shows some simple examples of the brace joint classification scheme.

Alternative classification strategies may be used, such as assigning classification in the order K-, X- and, finally, Y-joint response.



Key

g_1 gap 1 g_2 gap 2

Figure 14.2-2 — Examples of joint classification

Figure 14.2-2 h) is a good example of the axial force flow and classification hierarchy that should be adopted in the classification of braces in joints. The braces 1 and 2 on the left hand side of the chord act as a K-joint accounting for 50 % of the axial force in the diagonal brace. The other 50 % of the axial force in brace 1 forms an X-joint with brace 3. Replacement of brace axial forces by a combination of tension and compression forces to give the same net force is not permitted. For the example shown in Figure 14.2-2 h), replacing the axial force in brace 2 by a compression force of 1 000 and a tension force of 500 is not permitted, as this will result in an inappropriate X-joint classification for this horizontal brace and a full K-joint classification for brace 1.

Careful consideration should be given to determining the correct gap between braces in a K-joint. In Figure 14.2-2 a) the appropriate gap is between adjacent braces. However, if an intermediate brace exists, as in Figure 14.2-2 d), the appropriate gap is between the outer braces acting as the K-joint. In this case, since the gap is often large, the K-joint strength can revert to that of a Y-joint. Figure 14.2-2 e) is instructive in that the appropriate gap for brace 2 is g_2, whereas for brace 1 it is g_1. Although brace 3 is classified wholly as a K-joint (with brace 2 for 500 normal to the chord and with brace 1 for the remainder of the normal component of brace 3), the strength is determined by weighting the strength with gaps of g_1 and g_2 by the proportions of the axial force balancing from braces 1 and 2.

There are some instances where the joint behaviour is more difficult to define or is apparently worse than predicted using the above classification. Two of the more common cases in the latter category are associated with actions on a launch frame and with *in situ* actions on skirt pile-sleeves. Some guidance for such instances is given in A.14.2.4.

Once the breakdown according to axial brace force components is established, the strength of the joint can be determined using the procedures in 14.3.

14.2.5 Detailing practice

Joint detailing is an essential element of joint design. For unreinforced joints, the recommended detailing nomenclature and dimensioning are shown in Figures 14.2-3 and 14.2-4. Where an increased wall thickness or higher yield or toughness properties is required for the chord, this material should extend beyond the outside edge of incoming bracing by the greater of a minimum of one quarter of the chord diameter, or 300 mm. The strength of Y- and X-joints is a function of the can length (see 14.3.5) and short can lengths can lead to a reduction of the joint strength. Increasing the can lengths beyond the minimum values given here should be considered to avoid the need for downgrading strength.

When two or more tubulars join in an X-joint, the larger diameter member shall continue through the joint, and the other should frame onto the through member and be considered the minor member. Where members of equal diameter meet at an X-joint, it is more efficient to make the through member that which carries the greater forces. Unless specified otherwise on the drawings, when two or more minor members intersect or overlap at a joint, the order in which each member frames into the joint should be determined by wall thickness and/or diameter. The member with the thickest wall should be the continuous or through member, and the sequence for framing the remaining members shall be based on the order of decreasing wall thickness. If two or more members have the same wall thickness, the larger diameter member shall be the continuous or through member. If two or more members have the same diameter and wall thickness, either member may be the through member unless the designer has designated a through member. Sections of brace welds, which will be covered by other brace connections, shall be welded and the NDT (non-destructive testing) performed prior to cover up.

Where an increased brace wall thickness or higher yield or toughness properties is required for the brace, this material should extend beyond both the connection with the chord and the connection with any overlapping braces by the greater of a minimum of one brace diameter, or 600 mm.

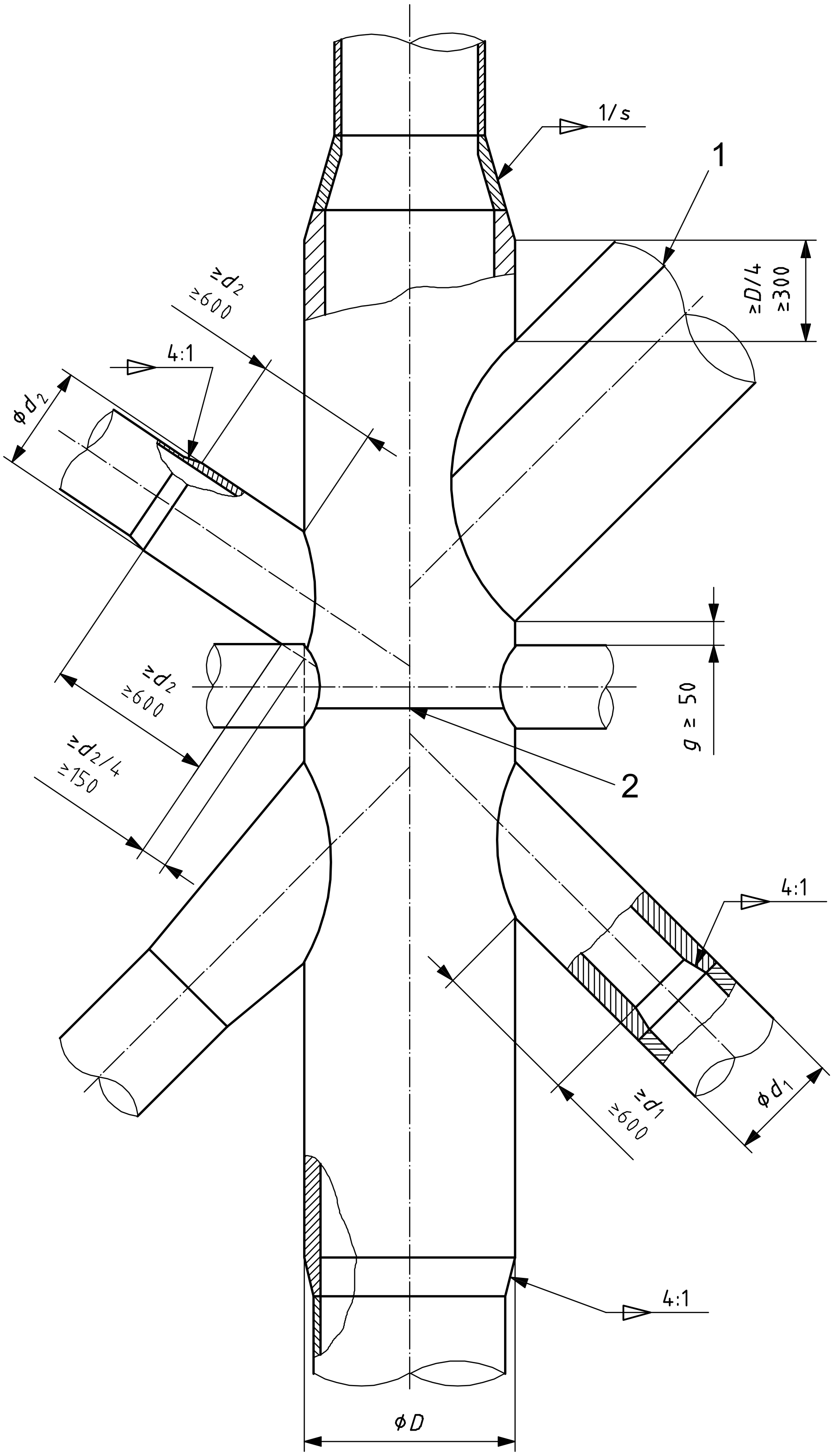
Neither the chord can nor the brace stub minimum dimensions given above include the length over which any thickness taper occurs; any difference in thickness between chord can and chord member or between brace stub and brace member shall be tapered at 1:4 or lower, see Figure 14.2-3. For joints where fatigue considerations are important, tapering on the inside can have both an undesirable influence on crack origin and make early detection of cracks more difficult; for such joints, tapering should be on the outside (i.e. matching internal diameters).

The nominal gap (i.e. excluding weld toes) between adjacent braces, whether in-plane or out-of-plane, should not be less than 50 mm. Overlapping of welds of non-overlapping braces at the weld toes shall be avoided. When braces overlap, the overlap should be at least $d/4$ (where d is the diameter of the through brace) or 150 mm, whichever is greater. This dimension is measured along the axis of the through member, see Figure 14.2-3.

Where braces overlap, the through brace shall have the thicker wall and shall be fully welded to the chord. Where there is a substantial overlap, the brace with the larger diameter should be the through member. The through brace can require an end stub to ensure that its thickness is at least equal to that of the overlapping brace.

Longitudinal seam welds and circumferential welds should be located to minimize or eliminate their impact on joint performance. The longitudinal seam weld of a brace should be located near the crown heel of the joint, see Figure 14.2-3. The longitudinal seam weld of the chord should be separated from incoming braces by at least 300 mm, see Figure 14.2-4. Where a chord requires a circumferential weld to achieve the desired can length, the weld should be positioned at a lightly loaded brace intersection, between saddle and crown locations, see Figure 14.2-3.

Dimensions in millimetres



Key

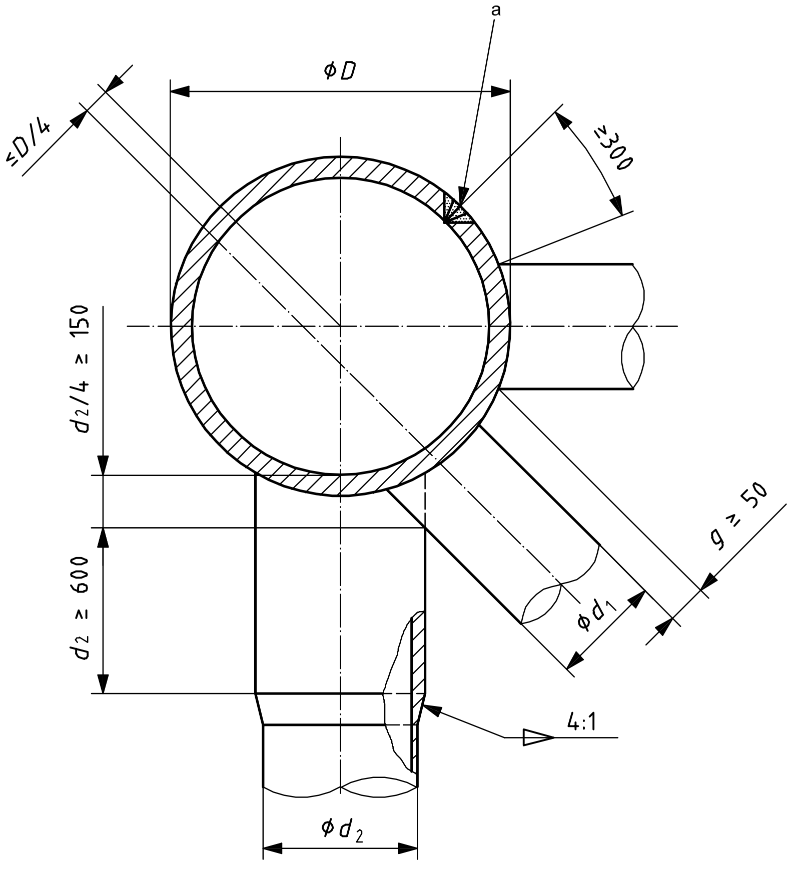
1 seam weld

2 can circumferential weld

s slope of conical transition ($s = 4$ for $D/T < 30$; flatter slope necessary for higher D/T ratios)

Figure 14.2-3 — In-plane joint detailing

Dimensions in millimetres



a Longitudinal seam weld of chord can.

Figure 14.2-4 — Out-of-plane joint detailing

14.3 Simple circular tubular joints

14.3.1 General

Simple tubular joints are joints having no gussets, diaphragms, grout or stiffeners. Simple Y- and X-joints have no overlap of principal braces, but simple K-joints may have overlaps up to 0,6 D.

The validity ranges for the formulae given in 14.3 are as follows:

$0{,}2 \leqslant \beta \leqslant 1{,}0$

$10 \leqslant \gamma \leqslant 50$

$30^\circ \leqslant \theta \leqslant 90^\circ$

$\tau \leqslant 1{,}0$

$f_y \leqslant 500\ \text{N/mm}^2$

For K-joints, the following validity range also applies:

$g\ T > -1{,}2\gamma$

Annex A discusses approaches that may be adopted for joints which fall outside the above range.

14.3.2 Basic joint strength

The strength of a joint varies not only with its materials and geometry but also with the pattern of forces on each brace. Consequently, these strengths can vary between load cases.

The strengths for simple tubular joints subjected to axial brace forces or moments only should be calculated for each brace, for each individual force component of tension, compression, in-plane bending and out-of-plane bending, and for each load case consisting of a combination of forces.

Representative strengths for simple tubular joints are given in Equations (14.3-1) and (14.3-2):

$$P_{uj} = \frac{f_y\, T^2}{\sin\theta} Q_u\, Q_f \tag{14.3-1}$$

$$M_{uj} = \frac{f_y\, T^2\, d}{\sin\theta} Q_u\, Q_f \tag{14.3-2}$$

where

P_{uj} is the representative joint axial strength, in force units;

M_{uj} is the representative joint bending moment strength, in moment units;

f_y is the representative yield strength of the chord member at the joint (SMYS or 0,8 of the tensile strength, if less), in stress units;

T is the chord wall thickness at the intersection with the brace;

d is the brace outside diameter;

θ is the included angle between brace and chord;

Q_u is a strength factor (see 14.3.3);

Q_f is a chord force factor (see 14.3.4).

For braces with a mixed classification, P_{uj} and M_{uj} should be calculated by weighting the contributions from Y-, K- and X-joint behaviour by the proportions of that behaviour in the joint. This means that P_{uj} and M_{uj} can be different for each load case considered, since joints can behave differently under different load cases, see A.14.3.2. However, for M_{uj} the values for Q_u for in-plane and out-of-plane moments are independent of the classification, see Table 14.3-1.

For joints with joint cans, P_{uj} shall not exceed the strength limits defined in 14.3.5.

The design strengths of simple tubular joints are

$$P_d = \frac{P_{uj}}{\gamma_{R,j}} \tag{14.3-3}$$

$$M_d = \frac{M_{uj}}{\gamma_{R,j}} \tag{14.3-4}$$

where

P_d is the design value of the joint axial strength, in force units;

M_d is the design value of the joint bending moment strength, in moment units;

$\gamma_{R,j}$ is the partial resistance factor for tubular joints, $\gamma_{R,j} = 1{,}05$.

14.3.3 Strength factor, Q_u

The strength factor, Q_u, varies with the joint classification and brace force type, as given in Table 14.3-1.

Table 14.3-1 — Values for Q_u

Joint classification	Brace force			
	Axial tension	Axial compression	In-plane bending	Out-of-plane bending
K	$(1{,}9+19\beta)Q_\beta^{0,5}Q_g$	$(1{,}9+19\beta)Q_\beta^{0,5}Q_g$	$4{,}5\ \beta\ \gamma^{0,5}$	$3{,}2\ \gamma^{(0,5\ \beta^2)}$
Y	$30\ \beta$	$(1{,}9+19\beta)Q_\beta^{0,5}$	$4{,}5\ \beta\ \gamma^{0,5}$	$3{,}2\ \gamma^{(0,5\ \beta^2)}$
X	$23\ \beta$ for $\beta \leqslant 0{,}9$ $20{,}7 + (\beta - 0{,}9)\ (17\ \gamma - 220)$ for $\beta > 0{,}9$	$\left[2{,}8+(12+0{,}1\gamma)\beta\right]Q_\beta$	$4{,}5\ \beta\ \gamma^{0,5}$	$3{,}2\ \gamma^{(0,5\ \beta^2)}$
Where Q_β and Q_g are given by Equations (14.3-5) to (14.3-8).				

The Q_u factor for tension forces for design is based on limiting the strength to first cracking. The Q_u factor that is associated with ultimate strength of Y- and X-joints for tension forces for use in assessment is given in A.14.3.3.

Q_β is a geometrical factor defined by

$$Q_\beta = \frac{0{,}3}{\beta(1-0{,}833\,\beta)} \qquad \text{for } \beta > 0{,}6 \qquad (14.3\text{-}5)$$

$$Q_\beta = 1{,}0 \qquad \text{for } \beta \leqslant 0{,}6 \qquad (14.3\text{-}6)$$

Q_g is a gap factor defined by

$$Q_g = 1{,}9 - 0{,}7\ \gamma^{-0,5}\ (g/T)^{0,5} \qquad \text{for } g/T \geqslant 2{,}0, \text{ but } Q_g \geqslant 1{,}0 \qquad (14.3\text{-}7)$$

$$Q_g = 0{,}13 + 0{,}65\ \phi\ \gamma^{+0,5} \qquad \text{for } g/T \leqslant -2{,}0 \qquad (14.3\text{-}8)$$

for $-2{,}0 < g/T < +2{,}0$, the gap factor, Q_g, may be found by linear interpolation between the results of Equations (14.3-7) and (14.3-8) for the limiting values of $g/T = -2{,}0$ and $g/T = +2{,}0$.

where

$\phi = t \times f_{y,b}/(T \times f_y)$

and, in addition to the definitions given in 14.3.2,

$f_{y,b}$ is the representative yield strength of the brace at the intersection with the chord, in stress units;

t is the brace wall thickness at the intersection with the chord.

14.3.4 Chord force factor, Q_f

The chord force factor, Q_f, is a factor that accounts for the presence of forces from factored actions in the chord:

$$Q_f = 1{,}0 - \lambda q_A^2 \tag{14.3-9}$$

where λ is a factor dependent on force pattern and

λ = 0,030 for brace axial force;

= 0,045 for brace in-plane bending moment;

= 0,021 for brace out-of-plane bending moment.

The parameter, q_A, is defined as follows:

$$q_A = \left[C_1 \left(\frac{P_C}{P_y} \right)^2 + C_2 \left(\frac{M_C}{M_p} \right)^2_{ipb} + C_2 \left(\frac{M_C}{M_p} \right)^2_{opb} \right]^{0,5} \gamma_{R,q} \tag{14.3-10}$$

where

P_C is the axial force in the chord member from factored actions;

M_C is the bending moment in the chord member from factored actions;

P_y is the representative axial strength due to yielding of the chord member not taking account of buckling, in force units

$$P_y = A f_y$$

f_y is the representative yield strength of the chord member, in stress units;

A is the cross-sectional area of the chord or chord can at the brace intersection;

M_p is the representative plastic moment strength of the chord member;

$\gamma_{R,q}$ is the partial resistance factor for yield strength, $\gamma_{R,q}$ = 1,05;

ipb refers to in-plane bending;

opb refers to out-of-plane bending;

C_1, C_2 are the coefficients given in Table 14.3-2.

Table 14.3-2 — Values for the coefficients C_1 and C_2

Joint type	C_1	C_2
Y-joints for calculating strength against brace axial forces	25	11
X-joints for calculating strength against brace axial forces	20	22
K-joints for calculating strength against balanced brace axial forces	14	43
All joints for calculating strength against brace moments	25	43

When calculating the chord force factor, Q_f, the higher value of q_A for the chord on either side of the brace intersection shall be used.

For K-joints, chord axial tension forces may be ignored when calculating Q_f.

14.3.5 Y- and X-joints with chord cans

For simple Y- and X-joints with a chord can, the joint representative axial strength shall be calculated using Equation (14.3-11):

$$P_{uj} = \left[r + (1-r)(T_n/T_c)^2 \right] P_{uj,c} \tag{14.3-11}$$

where

P_{uj} is the representative joint axial strength, in force units;

$P_{uj,c}$ is the value of P_{uj} from Equation (14.3-1), based on chord can geometrical and material properties, including Q_f calculated from chord can properties and dimensions;

r $= L_c/(2{,}5\ D)$ for joints with $\beta \leqslant 0{,}9$;

$= (4\ \beta - 3)\ L_c/(1{,}5\ D)$ for joints with $\beta > 0{,}9$;

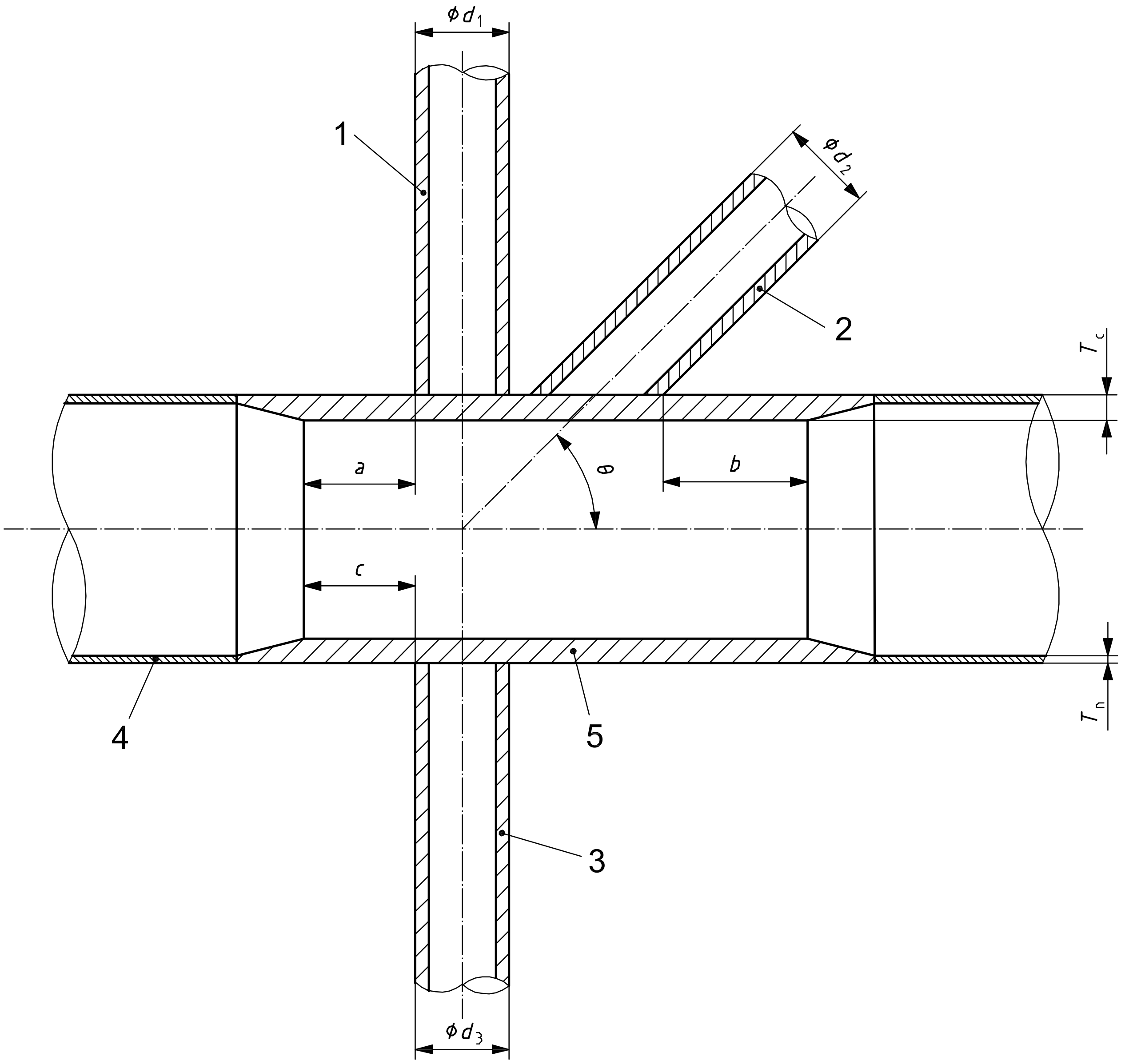
L_c is the effective total length, see Figure 14.3-1;

T_n is the lesser of the chord member thicknesses on either side of the joint, see Figure 14.3-1;

T_c is the chord can thickness, see Figure 14.3-1.

In no case shall r be taken as greater than unity. Figure 14.3-1 gives examples for the calculation of L_c.

Alternatively, an approximate closed ring analysis may be undertaken. Such an analysis should include plastic analysis with appropriate safety factors, and an effective chord length up to 1,25 D on either side of the line of action of the branch forces at the chord face, taking into account any thickness changes within this distance. Where multiple branches are in the same plane, and dominantly loaded in the same sense (tension or compression), the relevant perpendicular force is summed over all the braces on one side.



Key

1 brace 1
2 brace 2
3 brace 3
4 nominal chord
5 chord can

Calculation of effective total length

Brace	L_c
1	$2a + d_1$
2	$2b + d_2/\sin\theta$
3	$2c + d_3$

Figure 14.3-1 — Examples of chord length L_c calculation

14.3.6 Strength check

Each brace in a joint that is subjected either to an axial force or a bending moment alone, or to an axial force combined with bending moments, shall be designed to satisfy the following conditions:

$$U_j = \left|\frac{P_B}{P_d}\right| + \left(\frac{M_B}{M_d}\right)^2_{\text{ipb}} + \left|\frac{M_B}{M_d}\right|_{\text{opb}} \leqslant 1{,}0 \qquad \text{for all joints} \qquad (14.3\text{-}12)$$

$$U_j = \left|\frac{P_B}{P_d}\right| + \left(\frac{M_B}{M_d}\right)^2_{ipb} + \left|\frac{M_B}{M_d}\right|_{opb} \leqslant \frac{U_b}{\gamma_{zj}} \quad \text{for all joints except those identified as non-critical} \qquad (14.3\text{-}13)$$

where

U_j is the joint utilization;

P_B is the axial force in the brace member from factored actions;

M_B is the bending moment in the brace member from factored actions;

P_d is the design value of the joint axial strength (see 14.3.2);

M_d is the design value of the joint bending moment strength (see 14.3.2);

ipb represents in-plane bending moments and strengths;

opb represents out-of-plane bending moments and strengths;

U_b is the calculated brace utilization from the applicable brace interaction equation checks from Clause 13, the reduced limit on forces applies to critical joints only, see 14.2.3;

γ_{zj} is the extra partial resistance factor from Equation (14.2-2).

14.4 Overlapping circular tubular joints

Overlapping joints are joints where braces overlap in-plane or out-of-plane at the chord member surface. Figures 14.2-3 and 14.2-4 include both non-overlapping and overlapping braces.

The strength of joints that have in-plane overlap involving two or more braces may be determined using the requirements for simple joints defined in 14.3, with the following exceptions and additions.

a) Shearing of the brace parallel to the chord face is a potential failure mode and shall be checked.

b) Subclause 14.3.5 does not apply to overlapping joints.

c) If axial forces in the overlapping and through braces have the same sign (both in compression or both in tension), the check of the intersection strength of the through brace on the chord shall use the combined axial force representing the force in the through brace plus the portion of the overlapping brace force(s). The portion of the overlapping brace force may be calculated from the ratio of the cross-sectional area of the brace that bears onto the through brace to the full area of the overlapping brace.

d) For both in-plane or out-of-plane moments, the combined moments on the overlapping and through braces shall be used to check the intersection strength of the through brace on the chord. This combined moment shall account for the sign of the moments.

e) The overlap onto the through brace shall be checked by using the through brace as the chord in the equations in 14.3. The through brace strength shall also be checked for combined axial force and bending moment in the overlapping brace in accordance with 14.3.6 using the value of Q_f calculated for the through brace.

f) Where nominal thicknesses of the overlapping and through braces differ by more than 10 %, the thicker brace shall be the through brace.

Joints having out-of-plane overlap may be assessed on the same general basis as in-plane overlapping joints, except that the axial strength should normally revert to that for Y-joints.

Appendix 04

Efthymiou parametric equations for tubular joints

[2] (*Petroleum and Natural Gas Industry - Fixed steel offshore structure,* ISO 19902:2007, International Organization for Standardization, ISO), Pages 455-461.

For the α-Kellogg equations which are given in Reference [A.16.10-13], the EWI study concluded that they are generally more conservative than both the Efthymiou and the Lloyd's design SCF equations. Perhaps the most significant weakness of the α-Kellogg equations is that the predicted SCFs for all joint types are independent of β. This is clearly not the case as evidenced from test data and FEA results. Furthermore, the equations imply that chord SCFs are proportional to $\sqrt{\gamma}$ as opposed to observations which indicate that they increase linearly with γ. One advantage of the α - Kellogg equations is their simplicity.

In the comparison studies by Lloyd's Register, the Efthymiou SCF equations were found to provide a good fit to the screened SCF database, with a bias of about 10 % to 25 % on the conservative side [A.16.10-7]. They generally pass the HSE criteria for goodness-of-fit and conservatism. For the important case of K-joints under balanced axial forces, the Efthymiou equations did not pass the HSE criteria. A closer examination of this specific case revealed that these equations are satisfactory for both the chord and the brace side. For the chord side in particular, the Efthymiou equations provide the best fit to the database (COV = 19 %) and have a bias of 19 % on the conservative side. The second best equation (Lloyd’s) has a COV of 21 % and a bias of 41 % on the conservative side. The HSE criteria were deliberately designed to favour those equations that overpredict SCFs and to penalize underpredictions. This is the key reason why the Efthymiou equations for K-joints marginally failed the HSE criteria, even though they provide a good fit and also err on the conservative side.

The Lloyd’s design SCF equations generally pass the HSE criteria, except for T/Y-joints under axial force and ipb on the brace crown side. The reason for this is that there seems to be a systematic difference between the acrylic results and steel results for T/Y-joints at the brace crown.

Use of the Efthymiou SCF equations is recommended because this set of equations is considered to offer either the best option or a very good option for all joint types and types of brace forces and is the only set which covers overlapped K- and KT-joints.

Mix-and-match between different sets of equations is not recommended. The Efthymiou equations are also recommended in the 22nd edition of API RP2A WSD (see Reference [A.16.10-14]). The Efthymiou equations are given in Tables A.16.10-2 to A.16.10-5 and briefly discussed in A.16.10.2.2.2.

A.16.10.2.2.2 The Efthymiou equations

a) Overview

The Efthymiou equations cover SCFs in unstiffened T/Y-, X-, K- and KT-joints under all relevant brace force conditions. Overlapped K- and KT-joints are also covered. These expressions are based on extensive FEA using the PMBSHELL program. The program uses thick shell elements for modelling the chord and braces and 3-D brick elements for the welds. The weld profiles are as per AWS D1.1. This modelling enables direct extrapolation of stresses to the weld toes. The modelling and extrapolation removes the need for corrections, such as those attributed to Marshall [A.16.10-3], which were aimed at the brace side SCFs derived from thin shell FEA. Inclusion of the weld profiles with appropriate cut-back for high diameter ratios ensures realistic behaviour when modelling $\beta = 1{,}0$ joints.

b) T/Y-joints

For T/Y-joints under axial brace forces (see Table A.16.10-2) the SCFs are significantly influenced by the chord length and the fixity conditions at the ends of the chord. Beam bending of the chord influences primarily the crown SCFs, while for short chords ($\alpha < 12$), the shell distortion at the chord to brace intersection and hence the SCFs are affected by the fixity at the ends.

For beam bending the chord end fixity is defined by a parameter C, which is analogous to the effective length factor for buckling and has the range of 0,5–1,0. When $C = 0{,}5$, the ends are fully fixed and the equations degenerate to those of the fixed case. When $C = 1{,}0$ the chord ends are pinned. There are instances where beam bending of the chord is limited (see below). In such cases the chord length should be taken to be small (e.g. $\alpha = 12$ or less).

For joints with short chords ($\alpha < 12$), the correction factors, F_1 or F_2, in the Efthymiou equations should be applied when the ends of the chord are radially restrained. Factor F_1 is applicable when the ovalization at the chord ends is completely suppressed, for instance by a diaphragm, or if the ends are welded onto another stiff member.

c) X-joints

In X-joints under axial brace forces (see Table A.16.10-3) the SCFs at the saddles tend to dominate for all values of β, including the common case of $\beta = 1{,}0$. In joints with short chords the correction factors, F_1 or F_2, in the Efthymiou equations may be used, provided that ovalization of the ends is restrained to some extent. If the ends are completely free, the saddle SCFs should be increased. An approximate way of achieving this is to increase them by the ratio $1/F_2$.

The chord crown SCF (C_{X2}) under axial forces is derived from the corresponding SCF (C_{T2}) for T/Y-joints by suppressing beam bending of the chord, i.e. setting α to zero. For the brace crown SCF (C_{X4}), a better fit to the PMBSHELL database was obtained not by setting α to zero but by deleting completely the second term in the corresponding SCF (C_{T4}) for T/Y-joints.

d) K-joints

K-joints with a gap greater than one chord diameter ($\zeta > 1$) under balanced axial brace forces (Table A.16.10-4), should be classified as Y-joints for the purpose of SCF evaluation. The chord length parameter, α, should be set to 12 to reflect the fact that beam bending of the chord is limited. For opb when $\zeta > 1$, the SCF equations degenerate to the Y-joint equations and hence it is not necessary to re-classify these joints.

e) KT-joints

For KT-joints under balanced axial brace forces (see Table A.16.10-5) the SCFs on a diagonal brace are evaluated by considering the axial brace force to be balanced by the other diagonal brace, i.e. ignoring the central brace and hence degenerating the joint to a K-joint. For the central brace it is generally sufficient to consider that its axial force is balanced by one of the diagonal braces. If the diagonal braces are identical but the gaps differ, the maximum of the two gaps should be used, to be conservative. If the diagonal braces are not identical, then the central brace should be successively paired with each of the diagonal braces and the maximum resulting SCFs selected.

f) Validity ranges

The validity ranges for the Efthymiou equations are as follows:

$$\begin{array}{ccccc}
0{,}2 & \leqslant & \beta & \leqslant & 1{,}0 \\
0{,}2 & \leqslant & \tau & \leqslant & 1{,}0 \\
8 & \leqslant & \gamma & \leqslant & 32 \\
4 & \leqslant & \alpha & \leqslant & 40 \\
20^\circ & \leqslant & \theta & \leqslant & 90^\circ \\
\dfrac{-0{,}6\beta}{\sin\theta} & \leqslant & \zeta & \leqslant & 1{,}0
\end{array} \qquad \text{(A.16.10-3)}$$

For cases where one or more parameters fall outside this range, the following procedure should be adopted:

1) evaluate SCFs using the actual values of geometric parameters;

2) evaluate SCFs using the limit values of geometric parameters;

3) use the maximum of 1) or 2) above in the fatigue analysis.

Table A.16.10-2 — Equations for SCFs in T/Y-joints

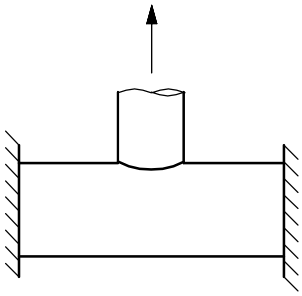
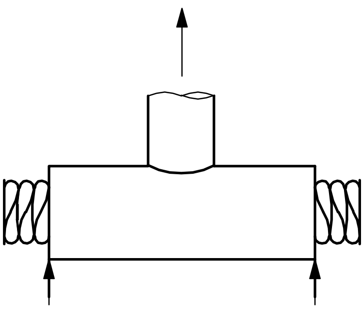
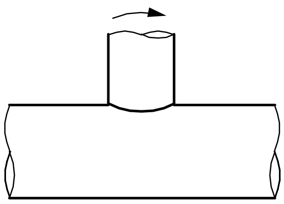
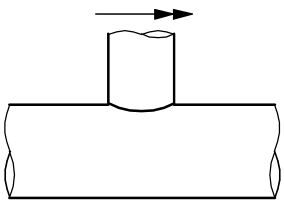
Type of brace force and fixity conditions	SCF equation
Axial brace force, chord ends fixed 	Chord saddle: $C_{CS} = F_1 \, C_{T1} \qquad C_{T1} = \gamma \, \tau^{1,1}\left[1,11 - 3(\beta - 0,52)^2\right]\sin^{1,6}\theta$ Chord crown: $C_{CC} = C_{T2} \qquad C_{T2} = \gamma^{0,2}\tau\left[2,65 + 5(\beta - 0,65)^2\right] + \tau\,\beta(0,25\alpha - 3)\sin\theta$ Brace saddle: $C_{BS} = F_1 \, C_{T3} \qquad C_{T3} = 1,3 + \gamma\,\tau^{0,52}\alpha^{0,1}\left[0,187 - 1,25\beta^{1,1}(\beta - 0,96)\right]\sin^{(2,7-0,01\alpha)}\theta$ Brace crown: $C_{BC} = C_{T4} \qquad C_{T4} = 3 + \gamma^{1,2}\left[0,12\exp(-4\beta) + 0,011\beta^2 - 0,045\right] + \beta\,\tau(0,1\alpha - 1,2)$
Axial brace force, general chord fixity 	Chord saddle: $C_{CS} = F_2 \, C_{T5} \qquad C_{T5} = C_{T1} + C_1(0,8\alpha - 6)\tau\beta^2\left(1 - \beta^2\right)^{0,5}\sin^2 2\theta$ Chord crown: $C_{CC} = C_{T6} \qquad C_{T6} = \gamma^{0,2}\tau\left[2,65 + 5(\beta - 0,65)^2\right] + \tau\,\beta(C_2\alpha - 3)\sin\theta$ Brace saddle: $C_{BS} = F_2 \, C_{T3}$ See above Brace crown: $C_{BC} = C_{T7} \qquad C_{T7} = 3 + \gamma^{1,2}\left[0,12\exp(-4\beta) + 0,011\beta^2 - 0,045\right] + \beta\tau(C_3\alpha - 1,2)$
In-plane bending (ipb) 	Chord crown: $C_{CC} = C_{T8} \qquad C_{T8} = 1,45\beta\tau^{0,85}\gamma^{(1-0,68\beta)}\sin^{0,7}\theta$ Brace crown: $C_{BC} = C_{T9} \qquad C_{T9} = 1 + 0,65\beta\tau^{0,4}\gamma^{(1,09-0,77\beta)}\sin^{(0,06\gamma-1,16)}\theta$
Out-of-plane bending (opb) 	Chord saddle: $C_{CS} = F_3 \, C_{T10} \qquad C_{T10} = \gamma\,\tau\,\beta\left(1,7 - 1,05\beta^3\right)\sin^{1,6}\theta$ Brace saddle: $C_{BS} = F_3 \, C_{T11} \qquad C_{T11} = \tau^{-0,54}\gamma^{-0,05}\left(0,99 - 0,47\beta + 0,08\beta^4\right)\times C_{T10}$

Table A.16.10-2 (*continued*)

Type of brace force and fixity conditions	SCF equation
	Short chord correction factors (α < 12): $F_1 = 1 - \left(0,83\beta - 0,56\beta^2 - 0,02\right)\gamma^{0,23}\exp\left(-0,21\gamma^{-1,16}\alpha^{2,5}\right)$ $F_2 = 1 - \left(1,43\beta - 0,97\beta^2 - 0,03\right)\gamma^{0,04}\exp\left(-0,71\gamma^{-1,38}\alpha^{2,5}\right)$ $F_3 = 1 - 0,55\beta^{1,8}\gamma^{0,16}\exp\left(-0,49\gamma^{-0,89}\alpha^{1,8}\right)$ where $\exp(x) = e^x$
	Chord-end fixity parameter, C: $0,5 \leqslant C \leqslant 1,0$ (Typically $C = 0,7$) $C_1 = 2(C - 0,5)$ $C_2 = C/2$ $C_3 = C/5$

Table A.16.10-3 — Equations for SCFs in X-joints

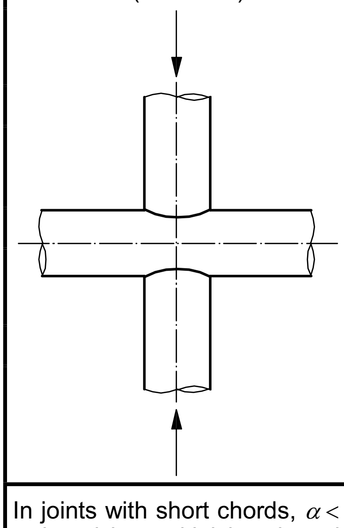
Type of brace force	SCF equation
Axial force (balanced) 	Chord saddle: $C_{CS} = C_{X1}$ $C_{X1} = 3,87\gamma\tau\beta\left(1,10 - \beta^{1,8}\right)\sin^{1,7}\theta$ Chord crown: $C_{CC} = C_{X2}$ $C_{X2} = \gamma^{0,2}\tau\left[2,65 + 5(\beta - 0,65)^2\right] - 3\,\tau\beta\sin\theta$ Brace saddle: $C_{BS} = C_{X3}$ $C_{X3} = 1 + 1,9\gamma\tau^{0,5}\beta^{0,9}\left(1,09 - \beta^{1,7}\right)\sin^{2,5}\theta$ Brace crown: $C_{BC} = C_{X4}$ $C_{X4} = 3 + \gamma^{1,2}\left[0,12\exp(-4\beta) + 0,011\beta^2 - 0,045\right]$
In joints with short chords, $\alpha < 12$, which have stiffened ends, both the chord saddle and the brace saddle SCF may be reduced by multiplying them by the short chord factor, F_1 or F_2. Factor F_1 can be used for stiff end reinforcements preventing ovalization as well as rotation of the chord wall, while factor F_2 can be used for end reinforcements partially preventing ovalization only. If the chord ends are completely free, both the chord saddle and the brace saddle SCF can increase significantly. An approximation can be obtained by increasing them by the ratio 1,0/F_2 (see A.16.10.2.2.2), but FEA is recommended. F_1 and F_2 are given in Table A.16.10-2.	

Table A.16.10-3 (*continued*)

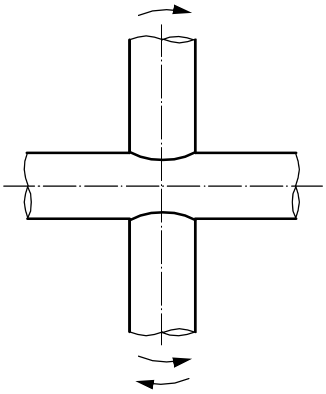
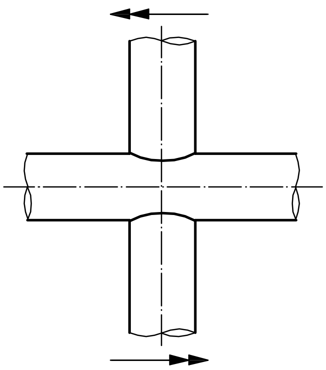
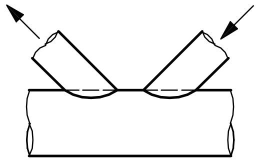
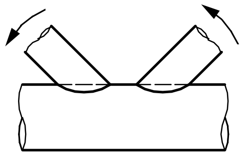
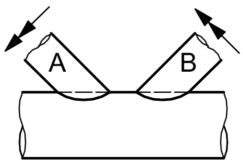
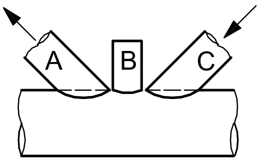
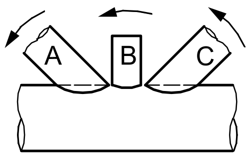
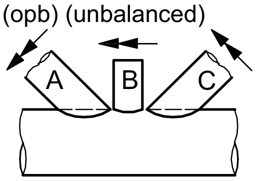
<table>
<tr><th>Type of brace force</th><th>SCF equation</th></tr>
<tr><td>In-plane bending (ipb)
(balanced or unbalanced)
</td><td>Chord crown:
$C_{\mathrm{CC}} = C_{\mathrm{T8}}$ see Table A.16.10-2
Brace crown:
$C_{\mathrm{BC}} = C_{\mathrm{T9}}$ see Table A.16.10-2</td></tr>
<tr><td>Out-of-plane bending (opb)
(balanced)
</td><td>Chord saddle:
$C_{\mathrm{CS}} = C_{\mathrm{X5}}$ $\quad C_{\mathrm{X5}} = \gamma\ \tau\ \beta\left(1{,}56 - 1{,}34\beta^{4}\right)\sin^{1{,}6}\theta$
Brace saddle:
$C_{\mathrm{BS}} = C_{\mathrm{X6}}$ $\quad C_{\mathrm{X6}} = \tau^{-0{,}54}\gamma^{-0{,}05}\left(0{,}99 - 0{,}47\beta + 0{,}08\beta^{4}\right)\times C_{\mathrm{X5}}$</td></tr>
<tr><td colspan="2">In joints with short chords, $\alpha < 12$, which have ends stiffened with a diaphragm or ring stiffener, both the chord saddle and the brace saddle SCF may be reduced by multiplying them by the short chord factor F_3; see Table A.16.10-2.
If the chord ends are completely free, saddle SCFs can increase significantly. An approximation can be obtained by increasing them by the ratio $1{,}0/F_3$ (see A.16.10.2.2.2), but FEA is recommended.</td></tr>
</table>

Table A.16.10-4 — Equations for SCFs in gap/overlap K-joints

Type of brace force	SCF equation
Axial forces (balanced) 	Chord: $C_C = C_{K1}$ $\quad C_{K1} = \left[\tau^{0,9}\gamma^{0,5}\left(0,67-\beta^2+1,16\beta\right)\sin\theta\right]\left(\dfrac{\sin\theta_{max}}{\sin\theta_{min}}\right)^{0,30}\times\left(\dfrac{\beta_{max}}{\beta_{min}}\right)^{0,30}$ $\times\left[1,64+0,29\beta^{-0,38}\arctan\left(8\zeta\right)\right]$ Brace: $C_B = C_{K2}$ $\quad C_{K2} = 1+C_{K1}\left(1,97-1,57\beta^{0,25}\right)\left(\tau^{-0,14}\sin^{0,7}\theta\right)+$ $\left[K\,\beta^{1,5}\gamma^{0,5}\tau^{-1,22}\sin^{1,8}\left(\theta_{max}+\theta_{min}\right)\right]\times\left[0,131-0,084\arctan\left(14\zeta+4,2\beta\right)\right]$ where $K = 0$ for gap joints; $K = 1$ for the through brace; $K = 0,5$ for the overlapping brace; the arctangents are evaluated in radians; τ, β, θ and the nominal stress relate to the brace being considered.
In-plane bending (ipb) (unbalanced) 	Chord crown, non-overlapping joint or overlap $\leqslant$ 30 % of contact length: $C_{CC} = C_{T8}$ see Table A.16.10-2 Chord crown, overlap > 30 % of contact length: $C_{CC} = 1,2\,C_{T8}$ see Table A.16.10-2 Brace crown, non-overlapping joint: $C_{BC} = C_{T9}$ see Table A.16.10-2 Brace crown, overlapping joint: $C_{BC} = C_{T9}\times\left(0,9+0,4\beta\right)$
Out-of-plane bending (opb) (unbalanced)[a] 	Chord saddle adjacent to brace A: $C_{CS} = F_4\,C_{K4}$ $\quad C_{K4} = C_{T10,A}\left[1-0,08\left(\beta_B\,\gamma\right)^{0,5}\exp\left(-0,8x\right)\right]$ $+C_{T10,B}\left[1-0,08\left(\beta_A\,\gamma\right)^{0,5}\exp\left(-0,8x\right)\right]\times\left[2,05\beta_{max}^{0,5}\exp\left(-1,3x\right)\right]$ where $x = 1+\dfrac{\zeta\sin\theta_A}{\beta_A}$ $C_{T10,A}$ and $C_{T10,B}$ (see Table A.16.10-2) are calculated with the parameters for braces A and B respectively. Brace saddle adjacent to brace A: $C_{BS} = F_4\,C_{K5}$ $\quad C_{K5} = \tau^{-0,54}\gamma^{-0,05}\left(0,99-0,47\beta+0,08\beta^4\right)\times C_{K4}$
	Short chord correction factor (α < 12) $F_4 = 1-1,07\beta^{1,88}\exp\left[-0,16\gamma^{-1,06}\alpha^{2,4}\right]$ where $\exp\left(x\right) = e^x$

[a] The designation of braces A and B is not geometry dependent. It is nominated by the user.

Table A.16.10-5 — Equations for SCFs in KT-joints

Type of brace force	SCF equation
Axial force (balanced)  	Chord: $C_\text{C} = C_\text{K1}$ see Table A.16.10-4 Brace: $C_\text{B} = C_\text{K2}$ see Table A.16.10-4 where $\zeta = \zeta_\text{AB} + \zeta_\text{BC} + \beta_\text{B}$ for the diagonal braces A and C; ζ = maximum of ζ_AB and ζ_BC for the central brace B.
In-plane bending (ipb)  	Chord crown: $C_\text{CC} = C_\text{T8}$ see Table A.16.10-2 Brace crown: $C_\text{BC} = C_\text{T9}$ see Table A.16.10-2
Out-of-plane bending (opb) (unbalanced)  	Chord saddle at diagonal brace A: $C_\text{CS} = C_\text{KT1}$ $C_\text{KT1} = C_\text{T10,A}\left[1-0{,}08(\beta_\text{B}\,\gamma)^{0{,}5}\exp(-0{,}8\,x_\text{AB})\right]\times\left[1-0{,}08(\beta_\text{C}\,\gamma)^{0{,}5}\exp(-0{,}8\,x_\text{AC})\right]$ $+\,C_\text{T10,B}\left[1-0{,}08(\beta_\text{A}\gamma)^{0{,}5}\exp(-0{,}8\,x_\text{AB})\right]\times\left[2{,}05\beta_\text{max}^{0{,}5}\exp(-1{,}3\,x_\text{AB})\right]$ $+\,C_\text{T10,C}\left[1-0{,}08(\beta_\text{A}\gamma)^{0{,}5}\exp(-0{,}8\,x_\text{AC})\right]\times\left[2{,}05\beta_\text{max}^{0{,}5}\exp(-1{,}3\,x_\text{AC})\right]$ where $x_\text{AB} = 1+\dfrac{\zeta_\text{AB}\sin\theta_\text{A}}{\beta_\text{A}}$ $x_\text{AC} = 1+\dfrac{(\zeta_\text{AB}+\zeta_\text{BC}+\beta_\text{B})\sin\theta_\text{A}}{\beta_\text{A}}$ Chord saddle at central brace B: $C_\text{CS} = C_\text{KT2}$ $C_\text{KT2} = C_\text{T10,B}\left[1-0{,}08(\beta_\text{A}\,\gamma)^{0{,}5}\exp(-0{,}8\,x_\text{AB})\right]^{(\beta_\text{A}/\beta_\text{B})^2}$ $\times\left[1-0{,}08(\beta_\text{C}\,\gamma)^{0{,}5}\exp(-0{,}8\,x_\text{BC})\right]^{(\beta_\text{C}/\beta_\text{B})^2}$ $+\,C_\text{T10,A}\left[1-0{,}08(\beta_\text{B}\,\gamma)^{0{,}5}\exp(-0{,}8\,x_\text{AB})\right]\times\left[2{,}05\beta_\text{max}^{0{,}5}\exp(-1{,}3\,x_\text{AB})\right]$ $+\,C_\text{T10,C}\left[1-0{,}08(\beta_\text{B}\,\gamma)^{0{,}5}\exp(-0{,}8\,x_\text{BC})\right]\times\left[2{,}05\beta_\text{max}^{0{,}5}\exp(-1{,}3\,x_\text{BC})\right]$ where $x_\text{AB} = 1+\dfrac{\zeta_\text{AB}\sin\theta_\text{B}}{\beta_\text{B}}$ $x_\text{BC} = 1+\dfrac{\zeta_\text{BC}\sin\theta_\text{B}}{\beta_\text{B}}$ Brace saddle: $C_\text{BS} = C_\text{KTB}$ $C_\text{KTB} = \tau^{-0{,}54}\gamma^{-0{,}05}\left(0{,}99-0{,}47\beta+0{,}08\beta^4\right)\times C_\text{CS}$

Appendix 05

Revised guidelines for maintenance and inspection of fire protection systems and appliances

[16] (*Revised Guidelines for the Maintenance and Inspection of Fire Protection Systems and Appliances*, MSC.1/Circ.1432, IMO).

INTERNATIONAL MARITIME ORGANIZATION

4 ALBERT EMBANKMENT
LONDON SE1 7SR
Telephone: +44 (0)20 7735 7611 Fax: +44 (0)20 7587 3210

MSC.1/Circ.1432
31 May 2012

REVISED GUIDELINES FOR THE MAINTENANCE AND INSPECTION OF FIRE PROTECTION SYSTEMS AND APPLIANCES

1 The Maritime Safety Committee, at its ninetieth session (16 to 25 May 2012), having considered a proposal by the Sub-Committee on Fire Protection, at its fifty-fifth session, and recognizing the need to include maintenance and inspection guidelines for the latest advancements in fire-protection systems and appliances, approved the Revised Guidelines for the maintenance and inspection of fire protection systems and appliances, as set out in the annex.

2 Member Governments are invited to apply the annexed Guidelines when performing maintenance, testing and inspections in accordance with SOLAS regulation II-2/14.2.2.1 on or after 31 May 2013 and bring the annexed Guidelines to the attention of shipowners, shipmasters, ships' officers and crew and all other parties concerned.

3 This circular supersedes MSC/Circ.850.

***

ANNEX

REVISED GUIDELINES FOR THE MAINTENANCE AND INSPECTION OF FIRE PROTECTION SYSTEMS AND APPLIANCES

1 Application

These Guidelines apply to all ships and provide the minimum recommended level of maintenance and inspections for fire protection systems and appliances. This information may be used as a basis for the ship's onboard maintenance plan required by SOLAS regulation II-2/14. These Guidelines do not address maintenance and inspection of fixed carbon dioxide systems or portable fire extinguishers. Refer to the comprehensive instructions provided in the Guidelines for the maintenance and inspections of fixed carbon dioxide fire-extinguishing systems (MSC.1/Circ.1318) for fixed carbon dioxide systems, and in the Improved Guidelines for marine portable fire extinguishers (resolution A.951(23)) for portable fire extinguishers.

2 Operational readiness

All fire protection systems and appliances should at all times be in good order and readily available for immediate use while the ship is in service. If a fire protection system is undergoing maintenance, testing or repair, then suitable arrangements should be made to ensure safety is not diminished through the provision of alternate fixed or portable fire protection equipment or other measures. The onboard maintenance plan should include provisions for this purpose.

3 Maintenance and testing

3.1 Onboard maintenance and inspections should be carried out in accordance with the ship's maintenance plan, which should include the minimum elements listed in sections 4 to 10 of these Guidelines.

3.2 Certain maintenance procedures and inspections may be performed by competent crew members who have completed an advanced fire-fighting training course, while others should be performed by persons specially trained in the maintenance of such systems. The onboard maintenance plan should indicate which parts of the recommended inspections and maintenance are to be completed by trained personnel.

3.3 Inspections should be carried out by the crew to ensure that the indicated weekly, monthly, quarterly, annual, two-year, five-year and ten-year actions are taken for the specified equipment, if provided. Records of the inspections should be carried on board the ship, or may be computer-based. In cases where the inspections and maintenance are carried out by trained service technicians other than the ship's crew, inspection reports should be provided at the completion of the testing.

3.4 In addition to the onboard maintenance and inspections stated in these Guidelines, manufacturer's maintenance and inspection guidelines should be followed.

3.5 Where particular arrangements create practical difficulties, alternative testing and maintenance procedures should be to the satisfaction of the Administration.

4 Weekly testing and inspections

4.1 Fixed fire detection and alarm systems

Verify all fire detection and fire alarm control panel indicators are functional by operating the lamp/indicator test switch.

4.2 Fixed gas fire-extinguishing systems

.1 verify all fixed fire-extinguishing system control panel indicators are functional by operating the lamp/indicator test switch; and

.2 verify all control/section valves are in the correct position.

4.3 Fire doors

Verify all fire door control panel indicators, if provided, are functional by operating the lamp/indicator switch.

4.4 Public address and general alarm systems

Verify all public address systems and general alarm systems are functioning properly.

4.5 Breathing apparatus

Examine all breathing apparatus and EEBD cylinder gauges to confirm they are in the correct pressure range.

4.6 Low-location lighting

Verify low-location lighting systems are functional by switching off normal lighting in selected locations.

4.7 Water mist, water spray and sprinkler systems

.1 verify all control panel indicators and alarms are functional;

.2 visually inspect pump unit and its fittings; and

.3 check the pump unit valve positions, if valves are not locked, as applicable.

5 Monthly testing and inspections

Monthly inspections should be carried out to ensure that the indicated actions are taken for the specified equipment.

5.1 Fire mains, fire pumps, hydrants, hoses and nozzles

.1 verify all fire hydrants, hose and nozzles are in place, properly arranged, and are in serviceable condition;

.2 operate all fire pumps to confirm that they continue to supply adequate pressure; and

.3 emergency fire pump fuel supply adequate, and heating system in satisfactory condition, if applicable.

5.2 Fixed gas fire-extinguishing systems

Verify containers/cylinders fitted with pressure gauges are in the proper range and the installation free from leakage.

5.3 Foam fire-extinguishing systems

Verify all control and section valves are in the proper open or closed position, and all pressure gauges are in the proper range.

5.4 Water mist, water spray and sprinkler systems

.1 verify all control, pump unit and section valves are in the proper open or closed position;

.2 verify sprinkler pressure tanks or other means have correct levels of water;

.3 test automatic starting arrangements on all system pumps so designed;

.4 verify all standby pressure and air/gas pressure gauges are within the proper pressure ranges; and

.5 test a selected sample of system section valves for flow and proper initiation of alarms.
(**Note –** The valves selected for testing should be chosen to ensure that all valves are tested within a one-year period.)

5.5 Firefighter's outfits

Verify lockers providing storage for fire-fighting equipment contain their full inventory and equipment is in serviceable condition.

5.6 Fixed dry chemical powder systems

Verify all control and section valves are in the proper open or closed position, and all pressure gauges are in the proper range.

5.7 Fixed aerosol extinguishing systems

.1 verify all electrical connections and/or manual operating stations are properly arranged, and are in proper condition; and

.2 verify the actuation system/control panel circuits are within manufacturer's specifications.

5.8 Portable foam applicators

Verify all portable foam applicators are in place, properly arranged, and are in proper condition.

5.9 Wheeled (mobile) fire extinguishers

Verify all extinguishers are in place, properly arranged, and are in proper condition.

5.10 Fixed fire detection and alarm systems

Test a sample of detectors and manual call points so that all devices have been tested within five years. For very large systems the sample size should be determined by the Administration.

6 Quarterly testing and inspections

Quarterly inspections should be carried out to ensure that the indicated actions are taken for the specified equipment:

6.1 Fire mains, fire pumps, hydrants, hoses and nozzles

Verify international shore connection(s) is in serviceable condition.

6.2 Foam fire-extinguishing systems

Verify the proper quantity of foam concentrate is provided in the foam system storage tank.

6.3 Ventilation systems and fire dampers

Test all fire dampers for local operation.

6.4 Fire doors

Test all fire doors located in main vertical zone bulkheads for local operation.

7 Annual testing and inspections

Annual inspections should be carried out to ensure that the indicated actions are taken for the specified equipment:

7.1 Fire mains, fire pumps, hydrants, hoses and nozzles

.1 visually inspect all accessible components for proper condition;

.2 flow test all fire pumps for proper pressure and capacity. Test emergency fire pump with isolation valves closed;

.3 test all hydrant valves for proper operation;

.4 pressure test a sample of fire hoses at the maximum fire main pressure, so that all fire hoses are tested within five years;

.5 verify all fire pump relief valves, if provided, are properly set;

.6 examine all filters/strainers to verify they are free of debris and contamination; and

.7 nozzle size/type correct, maintained and working.

7.2 Fixed fire detection and fire alarm systems

.1 test all fire detection systems and fire detection systems used to automatically release fire-extinguishing systems for proper operation, as appropriate;

.2 visually inspect all accessible detectors for evidence of tampering obstruction, etc., so that all detectors are inspected within one year; and

.3 test emergency power supply switchover.

7.3 Fixed gas fire-extinguishing systems

.1 visually inspect all accessible components for proper condition;

.2 externally examine all high pressure cylinders for evidence of damage or corrosion;

.3 check the hydrostatic test date of all storage containers;

.4 functionally test all fixed system audible and visual alarms;

.5 verify all control/section valves are in the correct position;

.6 check the connections of all pilot release piping and tubing for tightness;

.7 examine all flexible hoses in accordance with manufacturer's recommendations;

.8 test all fuel shut-off controls connected to fire-protection systems for proper operation;

.9 the boundaries of the protected space should be visually inspected to confirm that no modifications have been made to the enclosure that have created uncloseable openings that would render the system ineffective; and

.10 if cylinders are installed inside the protected space, verify the integrity of the double release lines inside the protected space, and check low pressure or circuit integrity monitors on release cabinet, as applicable.

7.4 Foam fire-extinguishing systems

.1 visually inspect all accessible components for proper condition;

.2 functionally test all fixed system audible alarms;

.3 flow test all water supply and foam pumps for proper pressure and capacity, and confirm flow at the required pressure in each section (Ensure all piping is thoroughly flushed with fresh water after service.);

.4 test all system cross connections to other sources of water supply for proper operation;

.5 verify all pump relief valves, if provided, are properly set;

.6 examine all filters/strainers to verify they are free of debris and contamination;

.7 verify all control/section valves are in the correct position;

.8 blow dry compressed air or nitrogen through the discharge piping or otherwise confirm the pipework and nozzles of high expansion foam systems are clear of any obstructions, debris and contamination. This may require the removal of nozzles, if applicable;

.9 take samples from all foam concentrates carried on board and subject them to the periodical control tests in MSC.1/Circ.1312, for low expansion foam, or MSC/Circ.670 for high expansion foam.
(**Note**: Except for non-alcohol resistant foam, the first test need not be conducted until 3 years after being supplied to the ship.); and

.10 test all fuel shut-off controls connected to fire-protection systems for proper operation.

7.5 Water mist, water spray and sprinkler systems

.1 verify proper operation of all water mist, water-spray and sprinkler systems using the test valves for each section;

.2 visually inspect all accessible components for proper condition;

.3 externally examine all high pressure cylinders for evidence of damage or corrosion;

.4 check the hydrostatic test date of all high pressure cylinders;

.5 functionally test all fixed system audible and visual alarms;

.6 flow test all pumps for proper pressure and capacity;

.7 test all antifreeze systems for adequate freeze protection;

.8 test all system cross connections to other sources of water supply for proper operation;

.9 verify all pump relief valves, if provided, are properly set;

.10 examine all filters/strainers to verify they are free of debris and contamination;

.11 verify all control/section valves are in the correct position;

.12 blow dry compressed air or nitrogen through the discharge piping of dry pipe systems, or otherwise confirm the pipework and nozzles are clear of any obstructions. This may require the removal of nozzles, if applicable;

.13 test emergency power supply switchover, where applicable;

.14 visually inspect all sprinklers focusing in areas where sprinklers are subject to aggressive atmosphere (like saunas, spas, kitchen areas) and subject to physical damage (like luggage handling areas, gyms, play rooms, etc.) so that all sprinklers are inspected within one year;

.15 check for any changes that may affect the system such as obstructions by ventilation ducts, pipes, etc.;

.16 test a minimum of one section in each open head water mist system by flowing water through the nozzles. The sections tested should be chosen so that all sections are tested within a five-year period; and

.17 test a minimum of two automatic sprinklers or automatic water mist nozzles for proper operation.

7.6 Ventilation systems and fire dampers

.1 test all fire dampers for remote operation;

.2 verify galley exhaust ducts and filters are free of grease build-up; and

.3 test all ventilation controls interconnected with fire-protection systems for proper operation.

7.7 Fire doors

Test all remotely controlled fire doors for proper release.

7.8 Breathing apparatus

.1 check breathing apparatus air recharging systems, if fitted, for air quality;

.2 check all breathing apparatus face masks and air demand valves are in serviceable condition; and

.3 check EEBDs according to maker's instructions.

7.9 Fixed dry chemical powder systems

.1 visually inspect all accessible components for proper condition;

.2 verify the pressure regulators are in proper order and within calibration; and

.3 agitate the dry chemical powder charge with nitrogen in accordance with system manufacturer's instructions.
(**Note**: Due to the powder's affinity for moisture, any nitrogen gas introduced for agitation must be moisture free.)

7.10 Fixed aerosol extinguishing systems

Verify condensed or dispersed aerosol generators have not exceeded their mandatory replacement date. Pneumatic or electric actuators should be demonstrated working, as far as practicable.

7.11 Portable foam applicators

.1 verify all portable foam applicators are set to the correct proportioning ratio for the foam concentrate supplied and the equipment is in proper order;

.2 verify all portable containers or portable tanks containing foam concentrate remain factory sealed, and the manufacturer's recommended service life interval has not been exceeded;

.3 portable containers or portable tanks containing foam concentrate, excluding protein based concentrates, less than 10 years old, that remain factory sealed can normally be accepted without the periodical foam control tests required in MSC.1/Circ.1312 being carried out;

.4 protein based foam concentrate portable containers and portable tanks should be thoroughly checked and, if more than five years old, the foam concentrate should be subjected to the periodical foam control tests required in MSC.1/Circ.1312, or renewed; and

.5 the foam concentrates of any non-sealed portable containers and portable tanks, and portable containers and portable tanks where production data is not documented, should be subjected to the periodical foam control tests required in MSC.1/Circ.1312.

7.12 Wheeled (mobile) fire extinguishers

.1 perform periodical inspections in accordance with the manufacturer's instructions;

.2 visually inspect all accessible components for proper condition;

.3 check the hydrostatic test date of each cylinder; and

.4 for dry powder extinguishers, invert extinguisher to ensure powder is agitated.

7.13 Galley and deep fat cooking fire-extinguishing systems

Check galley and deep fat cooking fire-extinguishing systems in accordance with the manufacturer's instructions.

8 Two-year testing and inspections

Two-year inspections should be carried out to ensure that the indicated actions are taken for the specified equipment.

8.1 Fixed gas fire-extinguishing systems

.1 all high pressure extinguishing agents cylinders and pilot cylinders should be weighed or have their contents verified by other reliable means to confirm that the available charge in each is above 95 per cent of the nominal charge. Cylinders containing less than 95 per cent of the nominal charge should be refilled; and

.2 blow dry compressed air or nitrogen through the discharge piping or otherwise confirm the pipe work and nozzles are clear of any obstructions. This may require the removal of nozzles, if applicable.

8.2 Fixed dry chemical powder systems

.1 blow dry nitrogen through the discharge piping to confirm that the pipe work and nozzles are clear of any obstructions;

.2 operationally test local and remote controls and section valves;

.3 verify the contents of propellant gas cylinders (including remote operating stations);

.4 test a sample of dry chemical powder for moisture content; and

.5 subject the powder containment vessel, safety valve and discharge hoses to a full working pressure test.

9 Five-year service

At least once every five years, the following inspections should be carried out for the specified equipment.

9.1 Fixed gas fire-extinguishing systems

Perform internal inspection of all control valves.

9.2 Foam fire-extinguishing systems

.1 perform internal inspection of all control valves;

.2 flush all high expansion foam system piping with fresh water, drain and purge with air;

.3 check all nozzles to prove they are clear of debris; and

.4 test all foam proportioners or other foam mixing devices to confirm that the mixing ratio tolerance is within +30 to -10% of the nominal mixing ratio defined by the system approval.

9.3 Water mist, water spray and sprinkler systems

.1 flush all ro-ro deck deluge system piping with water, drain and purge with air;

.2 perform internal inspection of all control/section valves; and

.3 check condition of any batteries, or renew in accordance with manufacturer's recommendations.

9.4 Breathing apparatus

Perform hydrostatic testing of all steel self-contained breathing apparatus cylinders. Aluminium and composite cylinders should be tested to the satisfaction of the Administration.

9.5 Low-location lighting

Test the luminance of all systems in accordance with the procedures in resolution A.752(18).

9.6 Wheeled (mobile) fire extinguishers

Visually examine at least one extinguisher of each type manufactured in the same year and kept on board.

10 Ten-year service

At least once every 10 years, the following inspections should be carried out for the specified equipment:

10.1 Fixed gas fire-extinguishing systems

.1 perform a hydrostatic test and internal examination of 10 per cent of the system's extinguishing agent and pilot cylinders. If one or more cylinders fail, a total of 50 per cent of the onboard cylinders should be tested. If further cylinders fail, all cylinders should be tested;

.2 flexible hoses should be replaced at the intervals recommended by the manufacturer and not exceeding every 10 years; and

.3 if permitted by the Administration, visual inspection and NDT (non-destructive testing) of halon cylinders may be performed in lieu of hydrostatic testing.

10.2 Water mist, water spray and sprinkler systems

Perform a hydrostatic test and internal examination for gas and water pressure cylinders according to flag Administration guidelines or, where these do not exist, EN 1968:2002 + A1.

10.3 Fixed dry chemical powder systems

Subject all powder containment vessels to hydrostatic or non-destructive testing carried out by an accredited service agent.

10.4 Fixed aerosol extinguishing systems

Condensed or dispersed aerosol generators to be renewed in accordance with manufacturer's recommendations.

10.5 Wheeled (mobile) fire extinguishers

All extinguishers together with propellant cartridges should be hydrostatically tested by specially trained persons in accordance with recognized standards or the manufacturer's instructions.

____

Hochschule für Angewandte Wissenschaften Hamburg
Hamburg University of Applied Sciences

Erklärung zur selbstständigen Bearbeitung einer Abschlussarbeit

Gemäß der Allgemeinen Prüfungs- und Studienordnung ist zusammen mit der Abschlussarbeit eine schriftliche Erklärung abzugeben, in der der Studierende bestätigt, dass die Abschlussarbeit „– bei einer Gruppenarbeit die entsprechend gekennzeichneten Teile der Arbeit [(§ 18 Abs. 1 APSO-TI-BM bzw. § 21 Abs. 1 APSO-INGI)] – ohne fremde Hilfe selbständig verfasst und nur die angegebenen Quellen und Hilfsmittel benutzt wurden. Wörtlich oder dem Sinn nach aus anderen Werken entnommene Stellen sind unter Angabe der Quellen kenntlich zu machen."

Quelle: § 16 Abs. 5 APSO-TI-BM bzw. § 15 Abs. 6 APSO-INGI

Dieses Blatt, mit der folgenden Erklärung, ist nach Fertigstellung der Abschlussarbeit durch den Studierenden auszufüllen und jeweils mit Originalunterschrift als letztes Blatt in das Prüfungsexemplar der Abschlussarbeit einzubinden.
Eine unrichtig abgegebene Erklärung kann -auch nachträglich- zur Ungültigkeit des Studienabschlusses führen.

Erklärung zur selbstständigen Bearbeitung der Arbeit

Hiermit versichere ich,

Name: Nguyen

Vorname: Quynh Chau

dass ich die vorliegende Bachelorarbeit bzw. bei einer Gruppenarbeit die entsprechend gekennzeichneten Teile der Arbeit – mit dem Thema:

Risk based methodology for planning periodic inspections of offshore substations in accordance with the standard for construction of the German Federal Maritime and Hydrographic Agency

ohne fremde Hilfe selbständig verfasst und nur die angegebenen Quellen und Hilfsmittel benutzt habe. Wörtlich oder dem Sinn nach aus anderen Werken entnommene Stellen sind unter Angabe der Quellen kenntlich gemacht.

- die folgende Aussage ist bei Gruppenarbeiten auszufüllen und entfällt bei Einzelarbeiten -

Die Kennzeichnung der von mir erstellten und verantworteten Teile der -bitte auswählen- ist erfolgt durch:

Hamburg	08.12.2015	
Ort	Datum	Unterschrift im Original