COMMITTEE V.1
ACCIDENTAL LIMIT STATES

COMMITTEE MANDATE

Concern for Accidental Limit States (ALS) of ship and offshore structures and their structural components during design. Types of accidents considered shall include fire, explosion, dropped object, collision and grounding. Attention shall be given to hazard identification and related risks, assessment of accidental loads and nonlinear structural consequences including residual strength. Uncertainties of ALS models for the use in design shall be highlighted. Consideration shall be given to practical application of design methods and to the development of ISSC guidance for implementation of ALS principles in engineering.

COMMITTEE MEMBERS

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KEYWORDS

Accidental Limit States, hazard identification, safety levels, accidental loads, accidental load effects, fires, explosions, dropped objects, ship collision, structural safety, material models for non-linear structural analysis, fire design benchmark study.

CONTENTS

Nomenclature:

ALARP	As Low As Reasonably Practical
CFD	Computational Fluid Dynamics
DAL	Dimensioning Accidental Loads
DOP	Dropped Object Protection
FEES	Fire, explosion and escape strategy according to ISO 13702
F&G	Fire and Gas
FW	Fire Water
FR	Functional Requirements (of the Contract)
HAZID	Hazard Identification Study
HC	Hydrocarbon
HSE	Health, Safety & Environment
LEL	Lower Explosion Limit
NFPA	National Fire Protection Association
PSA (PTIL)	Petroleum Safety Authority Norway
PFP	Passive Fire Protection
QRA	Quantitative Risk Analysis
TRA	Total Risk Analysis
TRR	Tubing Replacement Rig
TR	Temporary Refuge

Definitions:

Action: external load applied to the structure (direct action) or an imposed deformation or acceleration (indirect action). A hazard creates actions that are used to determine action effects from a hazard.

Accidental Action: load originated from identified hazards during the design phase; it is the outcome of QRA.

Action Effect: effect of action(s) on the structure or its components.

Design Accidental Load: chosen accidental load that is to be used as the basis for design.

   NOTE 1 The applied/chosen design accidental load may sometimes be the same as the dimensioning accidental load (DAL), but it may also be more conservative based on other input and considerations such as ALARP. Hence, the design accidental load may be more severe than the DAL.

   NOTE 2 The design accidental load should as a minimum be capable of resisting the dimensioning accidental load (DAL).

Dimensioning Accidental Event (DAE): Accidental events that serve as the basis for layout, dimensioning and use of installations and the activity at large.

Dimensioning Accidental Load (DAL): Most severe accidental load that the function or system shall be able to withstand during a required period of time, in order to meet the defined risk acceptance criteria.

   NOTE 1 DAL is normally defined based on DAE.

   NOTE 2 The dimensioning accidental load (DAL) is typically generated as a part of a risk assessment, while the design accidental load may be based on additional assessments and considerations.

   NOTE 3 The dimensioning accidental load (DAL) is typically established as the load that occurs with an annual probability of 10^{-4}.

Explosion load: time dependent pressure or drag forces generated by violent combustion of a flammable atmosphere.

Drag load: Drag force is caused by expanding hydrocarbon gas and air after explosion impinging upon an object. The drag force is a function of the fluid velocity and density along with the object's reference area and drag coefficient. The drag coefficient may further be a function of the Reynolds number. Reynolds number depends on the fluid density, viscosity, and velocity as well as the object's characteristic length. Smaller objects like piping which are inside an exploding gas cloud will be subjected to drag force.

Failure Strain: strain level at which the material is no longer providing any stiffness.

Fire load: Heat flux, normally defined in kW/m^2 for a specified duration.

Frequency: The number of measurements or (expected) observations having a certain value, or characteristic, during a certain observation period (e.g. expected annual frequency is a number of expected observations during one year observation period).

Hazard: potential for human injury, damage to the environment, damage to property, or a combination of these.

Hydrocarbon gas explosion: A process where combustion of a premixed gas cloud, i.e. fuel-air or fuel-oxidiser, is causing rapid increase of pressure. Gas explosions can occur inside process equipment or pipes, in buildings or offshore modules, in open process areas or in unconfined areas.

Integrity: the ability of a structure to perform its required function effectively and efficiently over a defined time period, while protecting health, safety and the environment.

Jet fire: Ignited release of pressurized, flammable gas and fluids.

Limit State: state beyond which the structure no longer fulfils the relevant assessment criteria.

Pool fire: Combustion of flammable or combustible fluids spilled and retained on a surface.
Probability: the relative frequency with which an event occurs, or is likely to occur

1.   INTRODUCTION

Accidents do happen. Structures like offshore facilities and ships are especially at risk as the presence of high-pressure gas and hydrocarbons involves high potential risk of leaks, explosions and fires. Heavy ship traffic, on the other hand, is associated with risk of ship impact or dropped objects on offshore structures, as well as collisions between the ships. Keeping in mind the risks involved, offshore and shipbuilding industries focus on diminishing accident hazards and protecting industrial facilities against them already in the design phase.

   Offshore facilities can only be put into operation based on national requirements. In order to obtain such permissions, offshore facilities have to be designed for safe operation. Therefore, design standards for offshore facilities include requirements and procedures for design against hazards. These standards describe methods for the assessment of hazards and actions, as well as structural consequences emerging from these hazards. In contrast to the requirements for offshore structures, ships have to obtain a class from a classification society in order to operate on international waters. To get certifications from classification societies, ships must be designed according to the classification rules and approved by the societies. Contrary to offshore industry, current ship design rules do not include requirements for design against hazards that may occur during ship operation.

   Since offshore industrial systems are designed using a number of different standards and procedures, special design tools have been established. Generally, two design philosophies are widely used in structural engineering; the working stress method WSM (allowable stress method ASM) and the load resistance design factors (LRDF). LRDF design method is based on the limit state concept, a concept that involves the idea that a structure has a limit state beyond which it does not perform its functions anymore. There are several limit states used in design that are meant to address hazards acting on industrial facilities. An example are Accidental Limit States, which have been developed as a guidance in design process. Accidental limit state for a structure defines a state beyond which the damage caused by accident inevitably leads to collapse as a result of domino effect. As long as resistance is an inherent property of the structure, loads rising from accidents are very uncertain and can be best presented as expected values or in the form of probabilistic models.

   This report is focused on the Accidental Limit States following Committee's conviction that defining ALS is a fundamental factor in the safe offshore and ship structure design. The scope of work includes hazard identification, fundamentals of ALS design, safety levels in ALS design and the assessment of accidental loads for hydrocarbon fires and explosions. A review of international and national standards for offshore structures and ships has been presented in the section on Fundamentals for ALS design. The

section on Safety Levels in ALS Design gives basis and evaluation of these levels, e.g. reliability based calibration of ALS codes. The Assessment of Accidental Loads section gives insight into methods of load assessment to be used in ALS design.

The Committee has also carried out a benchmark study. Its objective was to assess the capability and accuracy of available techniques for the prediction of structural response and strength of topside structures without and with Passive Fire Protection (PFP) when subjected to define fire loads. In the study the definition of the requirements for PFP using existing standards and numerical calculations are compared, and the capabilities of modern software to simulate fire loads and responses are evaluated.

2.   FUNDAMENTALS OF ALS DESIGN

2.1   Introduction

The limit state design (LSD), also known as the load and resistance factor design (LRFD), refers to a design method used in structural engineering. A limit state is a condition of a structure beyond which it no longer fulfills the relevant design criteria. The condition may refer to a degree of loading or other actions on the structure, while the criteria refer to structural integrity, fitness for use, durability or other design requirements. A structure designed by LSD is proportioned to sustain all actions likely to occur during its design life, and to remain fit for use, with an appropriate level of reliability for each limit state. Building codes based on LSD implicitly define the appropriate levels of reliability by their prescriptions.

From the viewpoint of a structural designer, four types of limit states are considered for steel structures, namely:

- serviceability limit state (SLS);
- ultimate limit state (ULS);
- fatigue limit state (FLS); and
- accidental limit state (ALS).

The primary aim of the ALS design for steel structures may be characterized by the following three broad objectives, namely:

- to avoid loss of life in the structure or the surrounding area;
- to avoid pollution of the environment; and
- to minimize loss of property or financial exposure.

In ALS design, it is necessary to achieve a design such that the main safety functions of the structure must not be impaired during any accidental event or within a certain time period after the accident. Since the structural damage characteristics and the behavior of damaged structures depend on the types of accidents, it is not straightforward to establish universally applicable structural design criteria for the ALS. Typically, for a given type of structure, design accidental scenarios and associated performance criteria must be decided upon the basis of risk assessment.

In many cases in design for accidental actions, prescriptive criteria derived from experience and related studies are used. Prescriptive criteria may be assumed accidental effects, such as damage extent due to collision, loss of a major bracing member, etc. Over the last two decades, the focus on ALS design criteria in the design requirements in different design standards has increased, especially within offshore structures. In order to replace or support the prescriptive criteria, there has been greater recognition of the use of risk based procedures for offshore structures where the consequences of structure failure are considered. Different approaches are used in different design standards and the complexity of the approaches is varying. Due to the development of high performance computers, more and more advanced methods such as nonlinear finite element analysis are used and consequently increase the reliability of the results.

ALS in design consists of several phases and a short overview is presented in a flowchart in Figure . The first step is the performance of a Quantitative Risk Assessment (QRA) which is a formalised specialist method for calculating individual, environmental, employee and public risk levels for comparison with regulatory risk criteria. In the design process action effects for every design hazard action are calculated in order to evaluate risk and to determine structural safety.
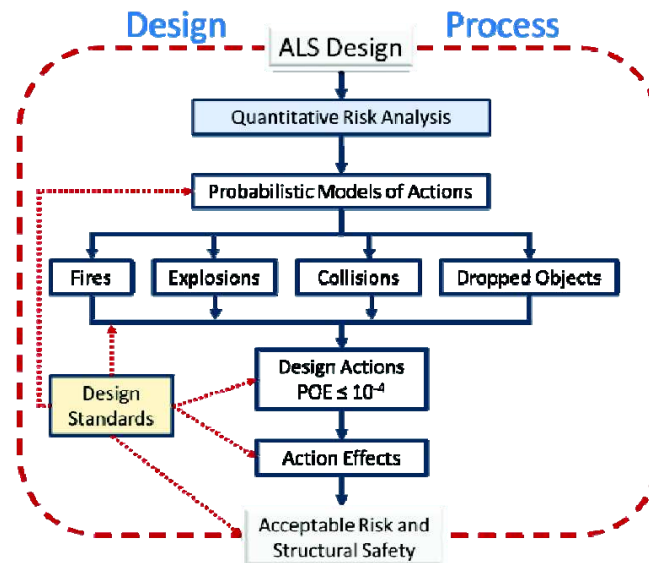
Figure 1. Flowchart for application of the ALS in Design.

2.2 Codes and standards

In design standards, it is required that damage from accidental actions with reasonable likelihood of occurrence shall not lead to complete loss of integrity of the structure, and the load-bearing function of the structures must be maintained. The design must be dimensioned such that critical parts for the overall strength are strong enough to withstand an accidental action, or alternatively, can be dimensioned in order to minimize the consequences with a certain redundancy without causing failure. For ALS design criteria, the most important standards for the offshore industry can be listed as:

- ISO 19900 series
  ISO 19900 General requirements for offshore structures
  ISO 19902 Fixed steel offshore structures
  ISO 19901-3 Topside structures
- Norsok standards
  N-001 Integrity of offshore structures
  N-003 Actions and action effects
  N-004 Design of steel structures
  Z-013 Risk and emergency preparedness assessment
- API Recommended Practice
  14J – Recommended Practice for Design and Hazards Analysis for Offshore Production Facilities, 2007
  2FB – Recommended practice for the design of offshore facilities against fire and blast loading, 2006
- DNV-GL standards
  DNV-RP-C204 Design against accidental load, 2010
  DNV-RP-C208 Determination of Structural Capacity by Non-linear FE analysis Methods, 2013
- ABS standard
  Accidental load analysis and design for offshore structures, 2013

The design standards provide an overview of approaches that can be used to identify and assess the effects of accidental structural loads arising from four accidental actions. These are dropped objects, vessel collision, fire and blast/explosion. Typically, the approaches in the design standards for these actions may either be determined by non-linear finite element analysis or energy considerations combined with simple elastic-plastic methods. Since the computer resources are increasing continuously, it is more and more common to use non-linear FE analysis.

A good introduction to the complete process may be obtained by studying the DNV Offshore standards series and Norsok Standards.

DNV-OS-A101 (DNV-OS-A101, 2014) provides a standard for the safety and arrangements of offshore installations by defining requirements for design loads, site arrangements, classification, shut

down logic, alarms, escape methods and communication. It serves as a contractual reference document between suppliers and purchasers and provides guidelines for designers, suppliers, purchasers and regulators as well as specifying procedures and requirements for units or installations subject to DNV classification. More specifically, it outlines simplistic Design principles and defines accidental loads; arrangements; hazardous area classification; emergency Shutdown (ESD) principles and requirements; escape and communication requirements for the design and operation of offshore structures.

The **DNV-OS-C101** (DNV-OS-C101, 2014) outlines the general design of offshore steel structures using LRFD methods and considers hazards that create accidental loads to include impact from ship collisions; impact from dropped objects; fires; explosions; abnormal environmental conditions; and accidental flooding. Their design principle considers that within their prescribed accidental limit states (ALS), structures with damage to their components arising from accidental events or operational failure can exceed ductile limits but that the ultimate resistance of damaged structures will maintain structural integrity after local damage or flooding with possible loss of station keeping (free drifting). However, DNV policy on accidental limit states notes that since the onus for design remains subject to the "inherent uncertainty of the frequency and magnitude of the accidental loads, as well as the approximate nature of the methods for determination of accidental load effects, that sound engineering judgement and pragmatic evaluations in the design are essential." Consequently, many initiatives remain underway to better define and reach agreement on reasonable design limit load definitions to establish rational standards.

DNV-RP-C204 (DNV-RP-C204, 2010) recommended practice for the analysis of loads against the offshore installation design so as to maintain the load-bearing function of structures during accidental events. The general philosophy is based on developing a design such that accidental loads will not impair the main safety functions of the installation. The explicit design guidelines and analysis methods presented by this standard enable very detailed and thorough design development. More specific requirements for probabilistic based methods to demonstrate structural reliability compliance with offshore standards are referred to DNV Classification Note 30.6 (DNV 1992). The standard also describes two other methods for the design of offshore steel structure including the Load and Resistance factor Design Method (LRFD) and a design method assisted by testing (quasi-deterministic). The design by load resistance factor method (LRFD) defines the basic variables as being the loads acting on the structure compared against the resistance of the structure. A target safety level is determined based on the anticipated variation in the load and resistance and the reduced probabilities that the loads will act simultaneously at their expected values. The level of safety of a structural element is considered to be satisfactory if the design load effect does not exceed the design resistance.

Germanicher Lloyd (GL, 2013) provides guidelines for hazard identification and risk assessment of offshore structures. They indicate hazard identification requires consideration of external hazards, such as ship collisions, extreme environmental conditions and helicopter crashes. The risk assessment of the hazards requires consideration of their tolerability to personnel, the facility and the environment. The assessment involves identifying the initiating events, the possibility of escalation, estimating their probability and their consequences.

Norsok standard N-001 (NORSOK-N-001, 2010) specifies general principles and guidelines for the design and assessment of offshore facilities, and the verification of load bearing structures subjected to foreseeable actions and related maritime systems. For accident limit state verification, the standard requires design checks, per ISO 19900, to ensure that the accidental action does not lead to complete loss of integrity or performance of the structure and related maritime systems. It requires that design actions and resistances be calculated using deterministic computational models with uncertainties in the computational models being covered by partial factors. However, the standard also allows design verification to be based on a more complete reliability design method, provided it can be documented that the method is suitable from a theoretical point of view, and that it provides adequate safety in typical known cases. This opens for use of reliability methods which entail calibration of action and material factors against a given failure probability level, or direct design by means of such methods. The safety level can be calibrated directly against the safety of known structure types and be based on corresponding assumptions. Direction on using reliability based design methods requires only that the results are on the safe side. It does not define any specific hazard or limit state requirement.

Norsok standard N-003 (NORSOK-N-003, 2007) defines permanent and variable actions pertinent to offshore structures. It provides a good overview of sea states, waves, wind, storms, ocean currents, ice, snow, earthquakes, temperature and their effects on structures. It defines accidental actions as those caused by abnormal operation or technical failure and includes, fires, explosions, ship collisions, dropped objects, helicopter crashes and changes to intended pressure differentials. Accidental assessments need to be supported with due account made for personnel qualifications, operational procedures, system design features, safety systems and control procedures. The accidental action design review applies to each accidental action that corresponds to an annual exceedance probability of 10^{-4}.

Detailed descriptions and considerations for appropriate assessment of each hazard is presented as well as methods to assign probabilistic values for combinations of events arising from the same phenomenon like hydrocarbon gas fires and explosions. The ALS design check requires evaluation of the structural damage caused by accidental actions; the ultimate capacity of structures with damage. The large uncertainties associated with determining the accidental actions, normally justify the utilisation of simplified nonlinear analyses methods both to calculate the damage and the global ultimate strength of the damaged structure. Such methods may be based on plastic mechanisms (yield hinge or line methods) with due recognition of possible premature rupture. Non-linear FE analyses are recognized to determine the ultimate capacity of damaged structures.

Norsok standard N-006 (NORSOK-N-006, 2009) provides detailed guidance on the selection and application of risk based and probabilistic methods for the design assessment of offshore structures, but does not address methods specific to hazards nor accidents.

Norsok standard Z-013 (NORSOK-Z-013, 2010) provides a very comprehensive description of risk and emergency preparedness analysis associated with exploration drilling, exploitation, production and transport of petroleum resources as well as all installations and vessels that take part in the activity.

2.3   Updates of codes and standards

The design standards are continuously updated in order to be more accurate and suited for today's accidental actions that may arise. As mentioned above, the trend is to use more advanced methods such as nonlinear finite element method. Recently, a new guideline on how to establish the structural resistance by the use of non-linear FE methods was developed (DNV-RP-C208, 2013). This guideline gives a relatively detailed description on FE modelling (material, mesh, boundary conditions, etc.), analysis procedure and post-processing of the results.

As a continuation of the development of DNV-RP-C208, a new Joint Industry Project was initiated in 2014 with the working title *'Nonlinear FE analysis of offshore structures – Further development and extension of design guidelines'*. This project will specifically address analyses related to accidental actions, especially what is required when considering high energy ship impacts. This is necessary since the acceptance criteria and analysis methodologies are specified in design standards written decades ago. These are normally based on representative offshore supply vessel (OSV) design in the order of 5000 tons displacement and characterized by a traditional raked bow. Modern OSVs are often designed with a bulbous bow and are up to 8000-10000 tons displacement with a consequent increase of the impact energy to be absorbed in the event of a collision. As a consequence, the Petroleum Safety Authority Norway is considering to increase recommended ship impact design energies (Kvitrud, 2013).

2.4   Uncertainties in ALS in Design

There are several uncertainties in ALS design due to the approximate nature of the methods for determination of action effects. It is therefore essential to apply sound engineering judgment in the design.

For determination of actions effects, the usual approach is either to use simple elastic-plastic methods or the non-linear FE analysis. Typically, the simple elastic-plastic methods are more conservative compared to the more sophisticated non-linear FE analysis, and these simple methods are typically not able to redistribute forces after failure of structural elements. On the other hand, the non-linear FE analysis is more complex and is able to redistribute forces without complete loss of integrity of the structure if a non-critical structural member fails. However, the non-linear FE analysis requires experienced users and there are several factors that must be accounted for in order to achieve reliable results. These factors can typically be material model, solution procedure, mesh refinement, etc. which are discussed in more detail in the consecutive sections. For detailed review of action effect assessment in ALS design see Sec. 6.

2.5   Practice for ships

Design incorporating ALS considerations has not been widely adopted by the shipping industry. This is largely due to the evolutionary nature of the industry over hundreds of years and the development rather of prescriptive rules for design; but also the challenges presented by being by definition structures which must move in different ocean regions and be exposed to a range of environmental loads that are not easy to quantify at the design stage. Nevertheless, there is growing interest in the use of alternative methods and this is also being pushed by IMO's long term target for Goal Based Standards; though this may be seen as reliability based safety limit approach rather than purely LSD.

It is therefore not surprising that there are few examples of whole ship design incorporating ALS in the literature. However, there are several examples of ship design where specific aspects of ALS design are considered.

Levander (2010) shows adoption of a range of safety features for the cruise liner Oasis of the Seas. A major design feature is that the ship itself should be its own best lifeboat, leading to adoption of a great deal of redundancy in the propulsion system, and location of prime movers in separate fire zones. These are methods that have been historically used for naval vessels. Damage stability of the liner was proven using probabilistic studies involving thousands of possible damage cases.

The specific design area of (damage through) collision and grounding is receiving considerable attention at the moment. Pedersen (2010) presents a review of available collision and grounding analysis procedures which supports the adoption of in risk based design methods. The mains steps are

- Estimation of the grounding and collision probability
- Models for calculation of the resulting grounding and collision damage
- Analysis of the conditions of the damaged vessels
- Estimation of costs associated with the accidents

Regarding the theoretical models, the classic approach is separate calculation of the main elements, namely the ship motion during the incident, the external dynamics, the structural response and the internal mechanics. Energy is transferred between these elements and damage estimated through a force-penetration curve. This approach suffers the drawback of requiring a priori knowledge of the penetration path – which is only easily estimated for simple right angle collisions where the struck ship is assumed motionless. The current research trend is for methods to couple two or more of these elements in the analysis; FEM may be used for the structural response.

Ehlers and Tabri (2012) present such a combined model which avoids direct use of FEM, a numerical/semi-analytical procedure for collision assessment including traffic density, striking angles and collision velocities, using the example of a RoPax vessel; that work is extended in Montewka et al. (2014).

Contributing to establishing statistics for reliability approach, Cho et al. (2014) made a parametric study of double hull tanker designs in collision resistance. Hussein and Soares (2009) also contribute in this area with a study of three double hull tankers designed from the IACS Common Structural Rules; they go on to argue the benefit of 20% increase in deck plate thickness in collision scenarios. Also Saydam and Frangopol (2013) use reliability methods to study the performance of a single skin oil tanker in sudden damage – specifically grounding and collision – scenarios. The effect of ship speed and heading to the waves, and hydrodynamic analysis based on the sea states are included; the operational conditions are shown to have significant effect.

Specifically regarding grounding damage, Kim et al. (2013) present a probabilistic design approach 'grounding damage index R-D diagram', where 'R' is residual strength and 'D' is damage index. This is based on the damage method proposed by Paik et al. (2012). In this study 50 grounding scenarios were modelled for VLCC, Suezmax, Aframax and Panamax oil tankers. This method is seen as a temporary first approach to the problem rather than a high quality model for design.

3.   HAZARD IDENTIFICATION

3.1   Introduction

Christou and Konstantinidou (2012) provide a good summary of existing databases from regulatory authorities concerning analysis of past accidents in offshore oil and gas operations including Hydrocarbon release, collisions and marine accidents including blowouts. Of particular interest, they report and quantify the nature of structural defects responsible for seven major offshore accidents between 1977 and 2010 and list lessons learned. It is apparent from a review of their findings and stated lessons learned, that the significant consequences, albeit possibly initiated by extreme loads (e.g. Alexander Kielland capsize 1980) were a direct result of the Ton Vrouwenvelder (2014) fifth column of human errors ranging from failure to inspect welds for cracks, failure to monitor for fatigue, improper drilling procedures (Ixtoc blowout 1979) (Adriatic Blowout 2004), miscommunication between shift workers (Piper Alpha explosion 1988), incorrect safety valve (Ekofisk Blowout 1977), improper cementing following pipe rupture (Montara Blowout 2009, Macondo blowout 2010). Their findings revealed a series of organization and management failures rather than deficiencies in structural design. Their recommendations stressed the need to address inadequate hazard identification procedures and separate the operational from regulatory body responsibilities.

Nevertheless, it remains important to better quantify and codify minimum design requirements for marine structures to withstand expected environmental and accidental loads. They include statistics from the DNV world offshore accident dataset (WOAD) to European and American ocean areas and note a lack of data from other areas. The highest numbers of incidents, in order, are related to FPSO/FSU Jackets, Jack-up and helicopter operations, followed by semi-submersibles, pipelines and well support structures.
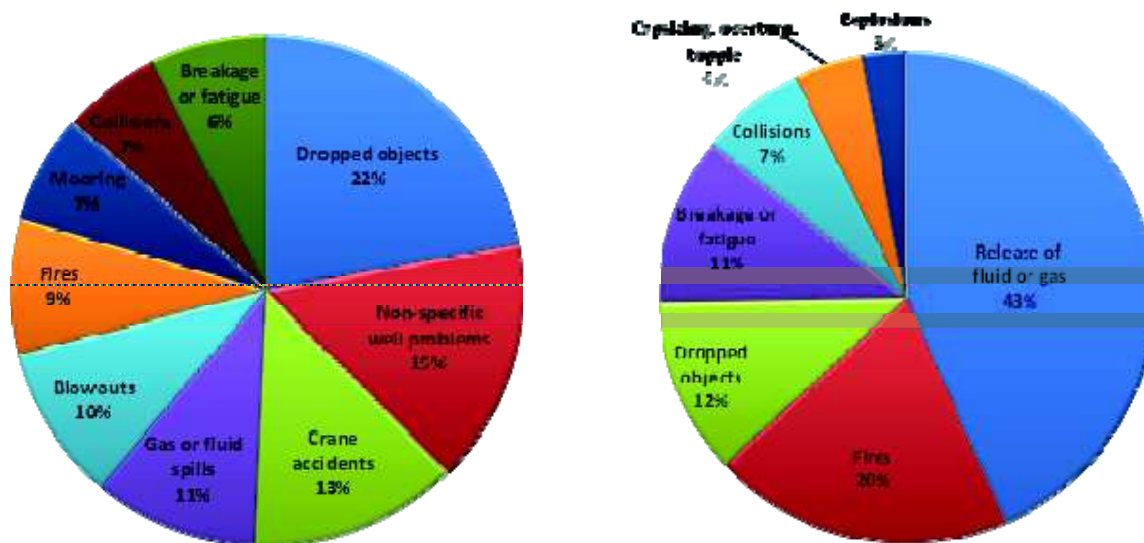
Figure 2. World accident statistics for drilling and production operations.

The WOAD statistics indicate that 86% (5323) accidents are non-human related but attributed to either unsafe procedures or the absence of procedure, with about 1 percent due to acts of war or sabotage and 8% attributed to some form of design defect.

The total losses during various accidents in offshore operations between 1970 and 2007 are given by OGP-434-17 (2010) and illustrated in Figure 2 Between 1970 and 2014 352 people were killed and 173 injured in fire and explosion accident associated with offshore operations.
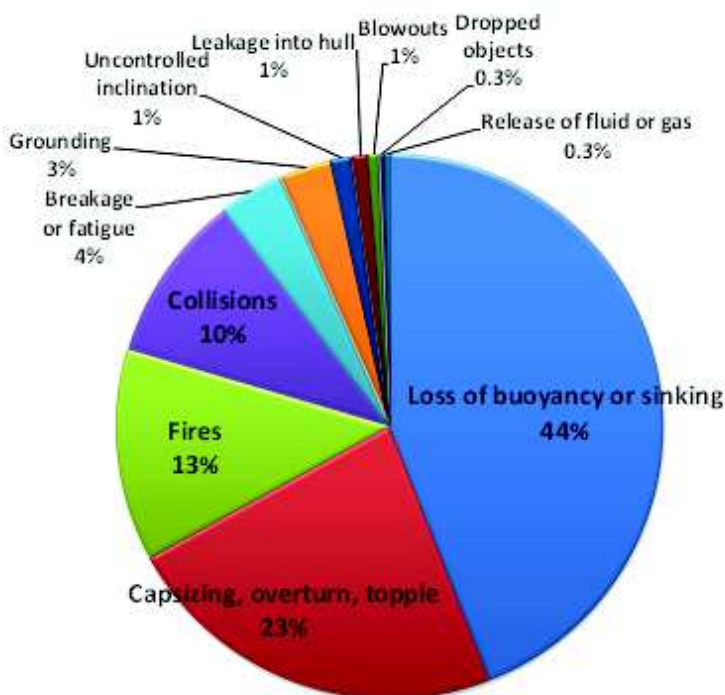
Figure 3. Percent distribution of total losses during offshore operation accidents between 1970 and 2007, OGP 2010.

Of additional interest, in cases of equipment malfunction, less than 0.18% was attributed to safety system malfunction or cataclysmic earthquakes or volcanic activity, with weather and fires responsible for about 25% respectively, 8% for structural failures and 34% from natural causes (wear-out etc.). Of additional

interest, fixed units tended to have three to four times the number of fluid or gas spills and fires than mobile units. In terms of total loss from a single cause, capsizing and loss of buoyancy (~80 respectively), followed by fire (41) and collisions (33).

The U.S. Bureau of Safety and Environmental Enforcement http://www.bsee.gov maintains a very useful database concerning details of offshore incidents and related statistics.

3.2   Hazard identification

Hazards refer to situations or events with the potential to cause human injury, damage to the environment or property. Operational hazards can take the form of monthly activities such as a shuttle tanker as it approaches an offshore facility, or daily activities like a crane that lifts equipment off the laydown area. What is distinct on the above hazards are the expected frequencies at which an offshore structure is exposed to these hazards, and the consequences on structural integrity they might have. In practice, the term "hazard" is often associated with probabilities of occurrence and expected consequences, the product of the two is defined as the risk.

A comprehensive hazard identification (HAZID) is often part of the wider update of the quantified risk assessment (QRA) and offshore safety case. It is defined by DNV (2002) as the process used to identify hazards as an essential first step in a risk assessment. Since hazards are diverse, there are many methods available for their determination. In general a structured approach is necessary to avoid overlooking possibilities and to encourage a creative identification of previously neglected scenarios. The process makes use of experience with accidents and draws from the expertise of a group of people with diverse backgrounds to avoid biasedness and benefit from brainstorming. The various techniques include hazard and operability studies (HAZOP) that apply a series of guidewords to systematically consider possible operational deviations from design intent. Failure mode effects and criticality analysis (FMECA) is another systematic method used to identify failure modes of electrical and mechanical systems. Failures are often rated critical if they have a high frequency or severity rating. The structured what-if technique (SWIFT) is similar to HAZOP but based on creative brainstorming rather than a formal list of guidewords.

Ultimately, identified hazards become part of a hazard register to support initial, and through-life safety management. In general, hazard assessments are best reported using a risk assessment matrix in line with qualitative or quantitative risk assessment. A typical matrix includes a severity scale, ranging from negligible, through critical to catastrophic, a frequency range from frequent, through probable, remote or incredible and a risk class that includes, intolerable, undesirable, and tolerable with or without mitigation measures. Variations of risk matrices are presented by ISO-17776 (2000).

Generally, Classification Rules require that structures comply with extreme permanent loads superimposed on regular environmental conditions, as well as for extreme environmental conditions superimposed on large permanent loads. The aim of Accidental Limit State (ALS) design is to provide adequate structural reliability to prevent catastrophic failure in the event of extreme and unusual environmental load conditions that arise from collisions, groundings, fires, explosions, system failures, dropped objects, terrorism and other cataclysmic events including earthquakes, hurricanes and tsunami's. Extreme and unusual environmental loads arise from events that are collectively considered to be hazards. In contrast to SLS, where design loads require elastic behaviour, materials subjected to action effects can be expected to deform beyond their elastic limits to the point of failure. The definition of actions and action effects in the context of ALS is given in Section 6. HAZID studies often comply with policies of various regulatory authorities and/or with the requirements set out in the operator's guideline on HAZID studies whereas available. It is the fundamental requirement for operators to perform an appropriate assessment to clearly demonstrate that envisaged risks have been identified and prevention measures put in place.

More specifically, in his overview of reliability based structural design, Ton Vrouwenvelder (2014) expands the list of hazards to broach five categories that include normal loads, natural accidents, man-made accidents, human influences and human errors. While it is possible to develop rational design for normal and accidental loads, and while human reliability analysis methods continue to develop, it is difficult to design against terrorism, vandalism, misuse, maintenance and operational mistakes. Particularly the hazard of terrorism was proved to be beyond operational hazards of marine structures, where even the recommended target reliability indexes by Pahos (2009) are dependent on the location of the panel and should be applied after a HAZID study. When applied to offshore structures, the requirement for the ALS is based on the philosophy that small damages, which inevitably occur, should not cause disproportionate consequences (Moan et al., 2002).

Given their diverse range of characteristics, absolute magnitudes of loading functions arising from accidents are based on the use of load exceedance curves. Such curves plot the magnitude of the load against the probability of exceeding it as a function of time. NORSOK-Z-013 (2010) provides a procedure to generate such load exceedance curves for offshore structures. Exceedance curves are intended to rationalize load limits. Figure 3 suggests a typical 10-fold increase in magnitude between the 1/100 year statistics compared to the 1/1000 year likely event. In the limit, such arguments are used to support establishing "rationalized" design requirements. The resultant accidental load that a facility can withstand during a required period of time is then called the "Dimensioning Accidental Load" or DAL.

The ABS (2013) guidance notes on accidental load analysis and design for offshore structures provide a design philosophy and hazard evaluation overview that considers four specific, non-environmental, hazards that need consideration in the design and operation of offshore structures and facilities. These include: i) ship collision hazards, ii) dropped object hazards, iii) fire hazards, and iv) blast hazards. In this context, ABS defines accidents to be: *"unintended events that arise during the course of installing, operating, or decommissioning an offshore oil and gas facility."* ABS describes the process for assessing accidental loads as one that follows traditional risk and structural assessment methods. Low risk and likelihood events, based on the owner's risk tolerance are removed and a refined assessment of the remaining accidents is undertaken to determine appropriate mitigation. They indicate that the structural performance criteria for each accident type need to be defined by the owners in keeping with requirements specified by regulatory, classification and corporate entities. These need to include the basis for localized structural member failure, such as yielding, buckling, plastic hinges, excessive deformation and connection failures; a global structural failure including hull girder collapse for SSFF, damaged stability and topside structural collapse; and consideration of safety critical elements including fire/blast containment walls, escape routes and muster areas and containment equipment failures.

Standard methods for identification of accident hazards are presented by API (2007), IMO (2002) Appendix 3, and ABS (2013). The guidance notes describe: "what-if" methods that use brainstorming techniques to postulate potential mishaps and ensure that appropriate safeguards are in place; Hazard and operability analysis (HAZOP) methods that use a systematic process to ensure that system design intent has appropriate performance safeguards; failure modes and effects analyses (FMEA) methods that use inductive reasoning to assess each system component failure mode and safeguard its effect on system performance; fault tree analysis (FTA) methods that use deductive analysis techniques to establish the relationship and consequences between human error, equipment failure and external events; and event tree analysis (ETA) methods using inductive analysis techniques to model possible outcomes of and event using decision trees to define multiple safeguards for events such as explosions and toxic releases.

Hazard curves like that shown in Figure 4 are used to describe the variation of hazard (loading) magnitude with the return period. The inverse of the return period (years) is referred to as the annual probability of exceedance. The 100 and 10,000 year returns correspond to the ISO-19900 (2013) ULS and abnormal ALS design events respectively. The slope of the curve presents the ratio between the 10,000 and 100 year return period. Accordingly, it provides a relative measure of the difficulty in reducing a given risk, in that the greater the slope the more challenging the hazard.
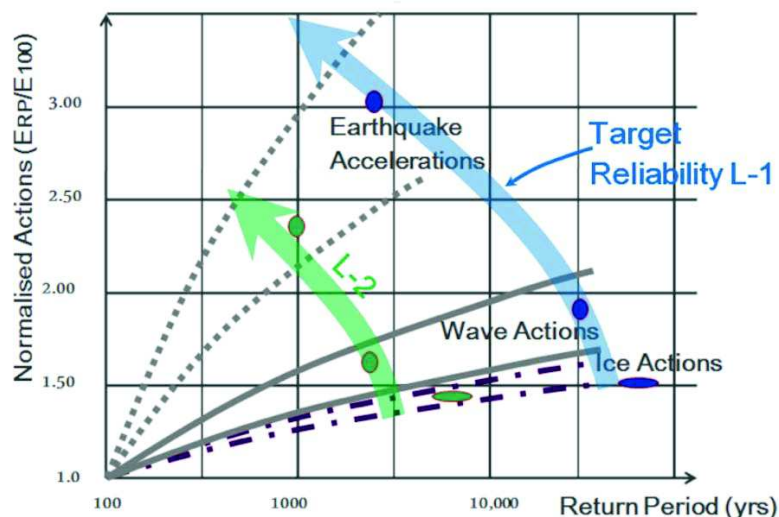

Figure 4. Hazard Curve (OGP-486, 2014).

In Figure 5, the slope for sea ice ranges between 1.4 and 1.5, wave actions between 1.5 and 1.9 and earthquakes ranging between 2.5 and 5. Iceberg phenomenon is grouped with earthquakes. The variation in the curves arises from regional differences, with northern North Sea curves typically forming the upper bounds compared to southern North Sea locations. Correspondingly, the hazards from earthquakes and icebergs are more challenging than wave loads, which in turn are more challenging than sea ice activity.

4.   SAFETY LEVELS IN ALS DESIGN

4.1   Introduction

Safety levels in ALS design are normally not given specifically. It is rather a *risk related design approach based on rare events that is applied*. Such risk based procedures are structure specific since ships and other floating installations, in addition to structural strength requirements, must fulfill requirements with respect to stability and capsizing. The structure must be designed so that it is robust in relation to the considered accidental actions by allowing local damage but preventing the collapse of the whole structure.

Additionally, ALS design must ensure that the installation is able to maintain structural integrity for a period of time under specified (usually reduced) environmental conditions sufficient to enable evacuation of personnel and avoid pollution.

4.2   Safety level of offshore structures in ALS

4.2.1   General

The load and resistance factor design codes, at least for offshore structures, specify limit states such as ULS, FLS, SLS and ALS to be considered in the design process. The safety level in such codes is normally calibrated by structural reliability analysis (SRA). Such calibrations are associated with a notional probability of failure. However, it should be noted that these formal probabilities of failure neither consider the consequences of accidents, nor provide a true measure of the actual probability of failure.

In more general terms accidental actions are associated with hazards due to errors in operation or system failure. They are typically influenced by human involvement in their cause and dominate the recorded risk picture of correctly designed offshore structures. A rational design against accidental hazards must involve and be based on quantitative risk analysis performed for the installation.

4.2.2   Discussion of new ISO standards for offshore structures

The new ISO standards for offshore structures, see e.g. ISO-19902 (2007) and ISO-19906 (2010), offer a practical implementation of the design approach against accidental and abnormal actions through the identification of relevant hazards and subsequent design using ALS criteria.

When using either the partial factor design format or a nonlinear pushover analysis, the designer quantifies the structure's ability to resist overload relatively to a chosen reference action, which depends on the limit state considered. In ISO-19902 (2007) the reference action is associated with either the 100-year environmental situation, or with accidental design situations with a significantly lower probability of occurrence. The first case is the regular design situation, and the second case represents accidental design situations.

All structures and their structural components must be designed to satisfy particular limit states. Each limit state is verified by defining a number of design situations, and requiring that the associated action effects shall meet given design criteria. It is noted that the acceptance criteria are met by demonstrating compliance with specified design procedures, not via a direct comparison with a target probability of failure.

Design situations are classified into three categories:

1. *persistent situations*, with a duration similar to the design life of the structure;
2. *transient situations*, with a much shorter duration and varying levels of intensity;
3. *accidental situations*, which are of short duration and low probability of occurrence.

Persistent and transient situations are typically considered in the regular ULS design situation using an environmental action with a return period of 100 years. Accidental situations consider hazards with a return period of 10 000 years in the ALS design situation.

4.2.3 Characterization of hazards

Accidental situations are treated by considering hazards. A hazard is any potential for human injury, damage to the environment, damage to property, or a combination of these. Using such a broad definition of a hazard, several of the usual hazards to a platform (e.g. extreme storms) are treated in the regular design situation, while hazards considered in accidental design situations are abnormal and accidental situations with a low, but not neglectable probability of occurrence.

In ISO-19902 (2007) hazards are categorized based on probability of occurrence and are divided into three main groups.

Group 1: hazards with a probability of occurring or being exceeded of the order of 10^{-2} per annum (return period of the order of 100 years);

Group 2: hazards with a 10 to 100 times lower probability of occurring or being exceeded, i.e. probabilities of the order of 10^{-3} to 10^{-4} per annum (return periods of the order of 1000 to 10 000 years);

Group 3: hazards with a probability of occurring or being exceeded markedly lower than 10^{-4} per annum (return periods well in excess of 10 000 years).

Designing for Group 1 hazards is normally treated by the regular design process and is incorporated in the verification of ULS limit states for persistent and transient design situations.

Other hazards belonging to Group 1 and not treated by the regular design process, as well as hazards belonging to Group 2, are addressed by a requirement that the structure shall satisfy particular accidental limit state (ALS) requirements.

Hazards falling into Group 3 are sometimes referred to as residual accidents and may normally be neglected in design.

4.2.4 Accidental design situations

ISO-19902 (2007) requires that a structure shall incorporate robustness through consideration of the effects of all hazards and their probabilities of occurrence to ensure that consequent damage is not disproportional to the cause. The intention of the associated ALS limit states is to ensure that the structure can tolerate specified accidental situations and, if damage occurs, that it subsequently maintains structural integrity for a sufficient period under specified environmental conditions to enable evacuation to take place. Accidental situations relate to two types of hazards:

Hazards associated with identified accidental events. These hazards belong to group 1 or group 2 and are not included in the regular design process.

Hazards associated with abnormal environmental actions. These hazards can occur due to very rare and abnormally severe environmental conditions. They correspond to a significantly longer return period than the ordinary design actions.

The two types of hazards are different by nature. In principle accidental events can in some cases be avoided by taking appropriate measures to eliminate the source of the event or by bypassing and overcoming its structural effects. In contrast to this, the possible occurrence of abnormal actions cannot be influenced by taking such measures.

An accidental design situation is considered in an accidental limit state (ALS) and normally comprises the occurrence of an identified accidental event or an abnormal environmental action, in combination with expected concurrent operating conditions and associated persistent and transient actions.

In ISO-19902 (2007) the regular design situation (ULS) for an L1 platform corresponds to an annual probability of $3*10^{-5}$ based on the reliability model given in Efthymiou et al. (1997), but ISO 19902 gives no direction for the determination of probabilities of failure associated with accidental situations.

After the issue of ISO-19902 (2007) the standard for arctic structures ISO-19906 (2010) was issued. This standard deals with abnormal ice action and calibrated the regular ULS design situation and the abnormal ALS ice action to the same safety level (10^{-5}/year) based on SRA approaches (OGP-434-16, 2010). They concluded that the safety level thus determined is of the same order of magnitude as the one used in ISO-19902 (2007).

4.2.5 ALS safety levels implied in structural codes

4.2.5.1 General

In a rational ALS criterion the accidental action should be defined as a characteristic value preferably defined in probabilistic terms. This has been done both in ISO-19902 (2007), ISO-19906 (2010) and in

NORSOK-N-003 (2007), where the characteristic accidental action for offshore structures is specified by an annual exceedance probability of 10^{-4}.

The ALS criterion also applies to abnormal environmental conditions such as hazards associated with abnormal environmental actions, e.g. wave actions. In this connection focus should also be given to abnormal waves with high crest or unusual shape – especially in such cases where the 10^{-2} wave might not reach the platform deck, but the 10^{-4} wave crest hits the deck and causes a significant increase in the wave loading.

4.2.5.2   *Introduction of risk based acceptance criteria*

The probability of failure associated with the accidental action *A* can be estimated as:

$$P_F = p(F \mid A) \cdot p(A) \tag{1}$$

where $p(F|A)$ is the conditional probability of failure given *A*, $p(A)$ is the probability of the accidental action, $p(F|A)$ is normally determined using either structural reliability analysis (SRA) or by Monte Carlo simulation, while $p(A)$ is determined using quantitative risk analysis (QRA).

In order to obtain a true reliability value by means of (1), both terms must be accurate reflections of reality, but generally any deficiencies in the determination of the two terms $p(A)$ and $p(F|A)$ makes it challenging to realistically estimate probabilities of failure for accidental actions.

Structural reliability analysis determines notional failure probabilities which can be quite different from real failure probabilities, and which should only be accepted as real probabilities of failure if they are calibrated to known conditions, and that is rarely the case. An alternative approach which has attracted increasing interest in recent years is to determine $p(F|A)$ by Monte Carlo simulation, as it may hold potential to provide a more accurate estimate of $p(F|A)$, provided a huge number of simulations are carried out. In addition, the determination of $p(A)$ is no trivial task as it ideally must consider all relevant failure causes and modes to closely reflect reality.

Therefore, (1) is at best believed to estimate an order of magnitude rather than a precise figure. Under ideal conditions the probabilities of failure determined by (1) should match actuarial values of failure collected over sufficiently long time of service for a given type of structure.

Ultimate failure consequences include fatalities, environmental damage and loss of assets, but most regulatory regimes have their main focus on limiting fatalities, and annual failure probabilities are favoured to ensure the same fatality risk of individuals at any time. Generally, it is accepted to treat different hazards and failure modes separately and assume the total failure probability equal to the sum of the individual probabilities.

Current offshore standards and regulatory regimes do not specify ALS acceptance criteria directly in probabilistic terms, and only ISO-19906 (2010) specifies acceptance criteria for ALS abnormal environmental action.

However, Moan (2009) provides some insight in the failure probability implied by the NORSOK-N-001 (2010) ALS criteria. The main focus is on fatalities, and acceptance criteria are derived based on acceptance of risk levels historically accepted by society. The risk model used, the FN diagram, was originally developed by Whitman (1981) based on early accident data for offshore structures and ships. The FN curve depicted below is based on more recent data. It compares experienced overall accident rates with respect to fatalities in the offshore and shipping industries. Floating platforms are not included because of limited experience with such platforms.
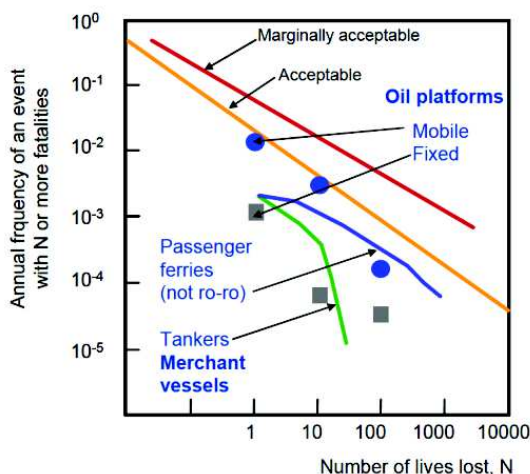

Figure 5. Frequency-fatality diagram (Whitman 2009).

In this diagram the horizontal axis represents the consequences in terms of fatalities while the vertical axis represents the annual occurrence rate of N or more fatalities. This diagram shows that the annual frequency of losses with 50-100 fatalities –which could be considered as total losses – is of the order of 6×10^{-5} for fixed (production) platforms and 10^{-3} for mobile units. Based on this data, the annual target failure probability of structural collapse due to each accidental action was chosen to be 10^{-5} for a fixed production platform.

Moan (2007) found that for structures designed to NORSOK requirements $p(F|A)$ is of the order 0.1. He thus concluded that the probability of failure due to an accidental event with an annual probability of 10^{-4} would be of the order $0.1 \times 10^{-4} = 10^{-5}$ per year. The steps in this ALS design approach are illustrated in Figure 6 below.
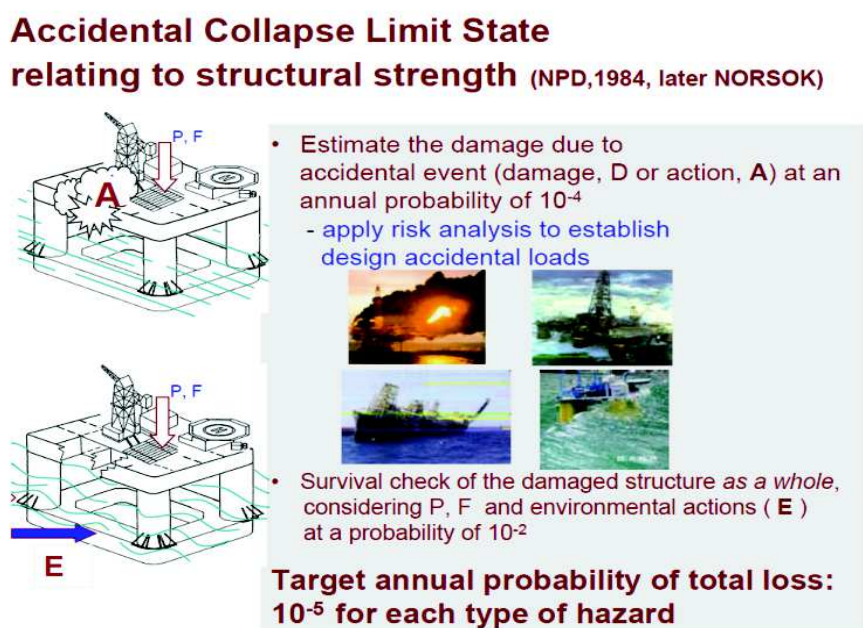

Figure 6. Norsok Accidental Limit State procedure (Moan, 2007).

DNV and ISO specify accidental and abnormal loads at an annual probability of 10^{-4}, but are not giving specific values for the annual probability of total loss.

4.3   *Safety level of Ship Structures in ALS*

4.3.1   *General*

Most of the past codes of ship structures have not been based entirely on the reliability analysis, neither on accident limit state requirements. But there is still a safety level implied in these rules. Teixeira and Soares (2010) discussed the reliability of intact and damaged ships and presented the relevant considerations that should be addressed when formulating the reliability problem of damaged ships. Particularly, the effect of the damage on the ship ultimate strength and the relevant changes on the still water and wave induced loads following damage, including the environmental conditions and duration of exposure that are applicable for the damaged ship were considered (Teixeira and Soares, 2010).

IMO proposed Goal Based Standard (GBS) requirements, meaning safety level requirements for ship structure design, which should reach the target safety level.

4.3.2   *GBS of Ship Structure Design*

After the 89th council of IMO in 2002, the GBS was implemented in the marine safety field. Later, the GBS framework with 5-tiers structures was presented by IMO and regarded as "standard of codes". Relating contents were included in previous ISSC reports.

The GBS has been developed in two parallel directions. One is safety level approach (SLA), which performs risk evaluation on current ship safety standards using overall analysis and formulates the risk acceptance criteria through formal safety assessment (FSA). The other is called deterministic approach, and formulates the quantitative functional requirements based on abundant practice experience on bulk

carriers and crude oil tankers. The 87th MSC adopted two documents that are important to the development of GBS: "International goal-based new ship construction standards for bulk carriers and oil tankers" and "Goal-based standards compliance validation guidelines". The former includes the general goals and 15 functional requirements of ship construction, corresponding to the first and second tier of the 5-tier structured GBS; while the latter explains validation standards and program of the 15 requirements in detail, corresponding to the third tier. The adoption of the two documents symbolized the breakthrough of the deterministic approach, indicating that GBS is operable in major ship domain. However, the risk level of requirements in GBS hasn't been evaluated and quantified, which means that the risk level of ship satisfying GBS is still unknown. Thus the current GBS is only based on historical experience instead of risk level.

In order to find sensible risk acceptance criteria and implement them into the GBS system, quantification of the risk level of current specification system needs to be accomplished via FSA. While this approach goes hand in hand with the development of FSA and historical statistics, due to the limitation of these two factors, the development of SLA will take a long time.

Following the implementation of GBS, IACS has identified gaps in the Common Structural Rules (CSR) for bulk carriers and tankers analysis that need to be mitigated. Recently, Harmonized Common Structural Rules (CSR-H) deal with the residual strength in this context, as no requirement to residual strength is given in the CSR. Some societies have their own criteria, e.g. (DNV-OS-A101, 2014) and (ABS, 1995b, ABS, 1995a), and these may be considered as potential requirements to be implemented in the CSR.

The residual strength after collision or grounding, including flooding, should be adequately covered by rule criteria, and in compliance with GBS.

4.3.3   Safety level in ULS in CSR

The hull girder ultimate capacity check is categorized as an ultimate limit state. The ultimate strength criterion is given in a partial safety factor format, and has been calibrated using structural reliability analysis techniques.

In the CSR, partial safety factors are calibrated for different target probability levels. A target level based on existing structures should be somewhere between 10^{-3} and 10^{-4}. A target level based on tabulated values, such as those used in DNV Classification Note 30.6 (DNV, 1992) as included in Table 1, indicates a stricter target level. In the CSR, the real failure probability of hull girder ultimate capacity for oil tankers is 10^{-3} (IACS, 2006).

Table 1. Acceptable Annual Probability of Failure (DNV, 1992).

Class of failure	Consequence of failure	
	Less serious	Serious
I-Redundant structure	10^{-3}	10^{-4}
II-Significant warning before the occurrence of failure in a non-redundant structure	10^{-4}	10^{-5}
III-No warning before the occurrence of failure in a non-redundant structure	10^{-5}	10^{-6}

4.3.4   Safety level in ALS in CSR-H

In CSR-H, the residual hull girder ultimate criterion is given in a partial safety factor format as well. It has also been calibrated using structural reliability analysis techniques. A comparative approach illustrated in Figure 7 was used. The ship following a collision or grounding damage should have the same safety level as intact ship for hull girder strength (Figure 7).

Two steps should be carried out to compare the safety level between intact ship and damaged ship (IACS, 2014):

(a) If the failure probability of the damaged case turns out to be lower than for the intact case, this would be an indication that a residual strength criterion is unnecessary. If a higher failure probability is obtained for the damaged case, this would indicate the need for a residual strength criterion.

(b) Irrespective of the results from (a), it is decided to have a residual strength criterion in the rules. The evaluations as illustrated and described in Figure 7 will be made to support the proposal of such a criterion. The intention is to make the residual strength criterion so that it corresponds to the same target reliability level as the intact criterion.
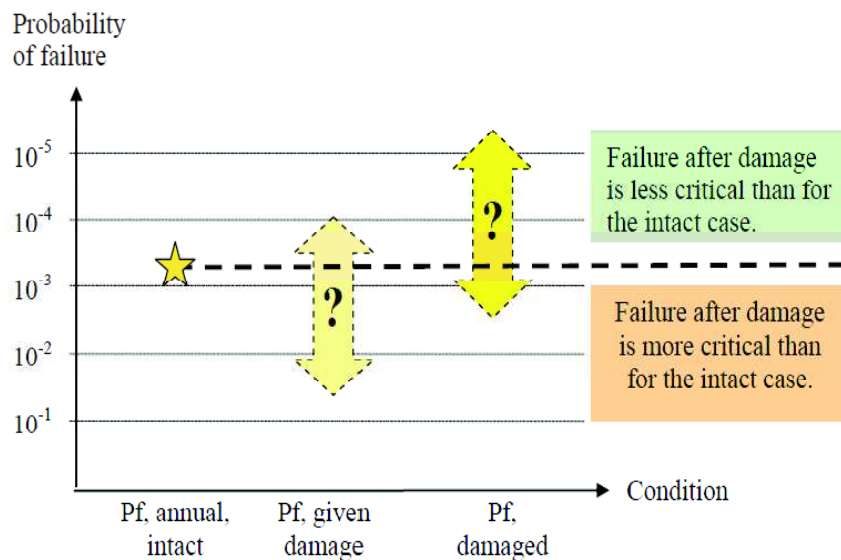
Figure 7. Comparative approach, structural reliability, intact and damaged condition (IACS, 2014).
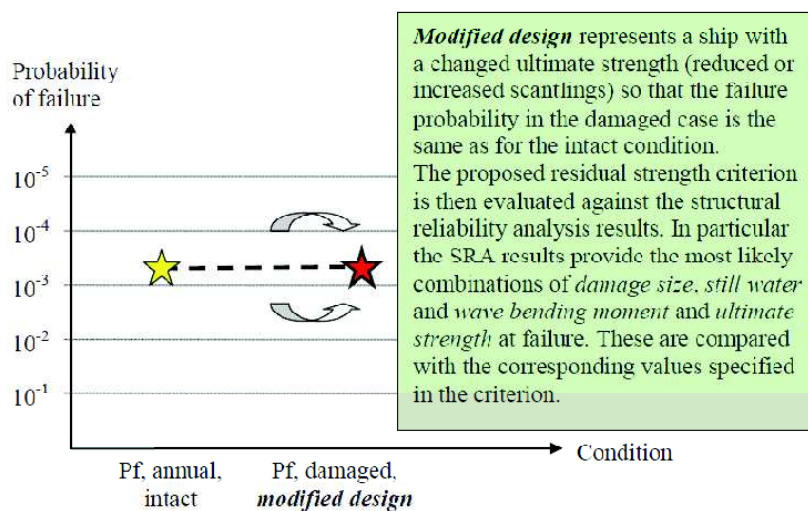
Figure 8. Principle for the development of a residual strength criterion (IACS, 2014).

IACS (2014) had made the comparison of reliabilities between the intact hull structure and the damaged hull structure. The hull girder strength of the bulk carrier was first adjusted to accurately meet the ultimate strength criterion in the rules. Then a structural reliability analysis, considering failure of the hull girder of an intact ship in North Atlantic environmental conditions, was performed. The failure probability could be calculated, following the same methodology as used for the development of the hull girder ultimate strength criterion for tankers (IACS, 2006). Thereafter, the probability of failure following collision was calculated for the same ship. Compared to the intact case, the following key items were accounted for:

- Annual probability of damage (collision)
- Probability of damage size, in the case of collision
- Reduction in capacity as a function of damage size
- Increase in still water bending moment due to damage
- Environmental conditions when the damage occurs
- Exposure time to environment, before rescue to shore

The results in (IACS, 2014) show that the annual probability of failure is 8×10^{-4} for the intact case. This compares well with the results obtained for tankers (IACS, 2006). It is seen that the failure probability during three months in the so-called collision environment is higher (5×10^{-3}) when calculated conditional on the event of collision. Assuming that the annual probability of a collision event is 0.01, the

annual probability of hull girder failure following collision damage is then 5×10^{-5}. This is lower than the failure probability for the intact case. Now the residual strength criterion will be considered in isolation, and adjusted to meet the same target reliability level of 8×10^{-4} from the intact case. This is done by redesigning the hull girder as illustrated in Figure 9, and is implemented by reducing the capacity until the target failure probability is obtained.

5.   ASSESSMENT OF ACCIDENTAL LOADS

5.1   Introduction

The subject of this chapter is the assessment of accidental loads which are implemented in the limit state based structural design. It is not only the calculation procedures of accidental loads but also determination procedures of the design accidental loads that are described in this chapter.

The characteristics of accidental loads and various methods to calculate them, that is, theoretical or empirical formulae, FEA (Finite Element Analysis), CFD (Computational Fluid Mechanics), and so on, were well described in previous ISSC reports. Collision and grounding accidents for ship structures are found in ISSC (2006), and explosion, fire, dropped object and collision accidents for offshore structures in ISSC (2012). Therefore this report focuses on the research and new findings in the field of accidental loads presented in recent literature.

In practical Accidental Limit State (ALS) design as well as other limit state designs, the design loads need to be determined methodically and effectively, taking into account the degree of accuracy and economic feasibility, as well as safety inherent in the each design phase. It is impossible to guarantee absolutely safe structures at the design stage because of numerous uncertainties, such as loadings, materials, structural behavior or human and environmental factors. An accident is usually a chain of unpredictable events and often a multi-physics phenomenon. The challenges, however, do not free designers from the responsibility of minimizing potential risks. Therefore, ALS design requires the design concepts based on risk assessment and probabilistic approach.

Recently, the risk based probabilistic assessment for accidental loads is increasingly being used and continuously developed, especially in the design of offshore facilities. That is, the state-of-the-art concept in determining design accidental loads is the probabilistic approach rather than the deterministic one which is usually based on the worst-case scenario. Therefore, the design standards and rules which are practically used in the ALS design of offshore are summarized in this chapter. Additionally, the generally accepted practices of industries for risk based or probabilistic determination of design loads are introduced. Identification of accidental situations or actions to be considered in the ALS design can be found in the relevant design standards and rules. Collision, dropped objects, explosion and fire accidents for offshore structures are clearly mentioned as accidental design cases in ISO, API, NORSOK standard or offshore codes from class societies (DNV, ABS, LR, BV, etc.). The specific description of accidental actions for ship structures is hardly found in the ship rules of class societies. Therefore, only the above accidental actions are considered in this chapter.

5.2   Explosion Load Assessment

The safety assessment of offshore facilities against explosion accidents should be performed in the design phase to prevent loss of lives or catastrophic failure of structures, and be preceded by the process of determining the design accidental load for explosion.

The design explosion load can be derived by the worst-case scenario, and the results are often too large to be accommodated by the facilities. Therefore, some reasonable and practicable level of loads to assist in determining design load is used. The Dimensional Accidental Load (DAL) which is usually established as the load that occurs with an annual exceedance frequency of 10^{-4} , as well as two levels of explosion loading, that is Ductility Level Blast and Strength Level Blast (DLB and SLB) loads were suggested by some regulation authorities (API, 2006, Oil&GasUK, 2007, NORSOK-Z-013, 2010). These are now widely used in offshore projects and well explained in ISSC (2012).

There are some references for the derivation of accidental loads. For instance, DNV-OS-A101 (2014) defines generic design loads for some accidental cases, and API (2006) refers to nominal loads and empirical formulas for fire and explosion. NORSOK-Z-013 (2010) recommends the probabilistic approach with CFD simulation for explosion, and similarly FABIG (2014) recommends the probabilistic CFD simulation for fire. Oil&GasUK (2007) and FABIG (1992) including its Technical Notes are an overall guidance for explosion and fire engineering. However, in offshore projects client requirements or specification of the facilities and best practices of industry developed from engineering works are commonly used.

Explosion loading scenarios are numerous and specific to the facility or calculation model, and usually developed on the base of risk analysis which is already described in section 2, 3, and 4 of this report. The

explosion load can be calculated by the nominal load method, simplified method based on theoretical formulas, and numerical simulation.

5.2.1 *Deterministic approach*

The easiest way to get the design accidental loads is to bring the prescriptive loads by referring to the relevant rules, standards or industrial guidance. The nominal load is a space averaged, peak explosion overpressure for typical concept types of installations, and based on limited design data set and operating experiences (API, 2006, DNV-OS-A101, 2014). Therefore, it is often a conservative value and used in the early project phase.

In the case of simplified calculation models for explosion loads, there are some empirical models such as TNT method for the high explosive and Multi-energy methods (Lees, 1996) or B-S-T model (Tang and Baker, 1999) for vapor cloud explosion. These are based on correlations with experimental data, and usually used to predict far field blast effects. Another simplified calculation model is the phenomenological method, which is based on a physical process more than the empirical model and has better accuracy with consideration of actual geometry using a simplified system. However, these simplified models are gradually substituted by the numerical simulation mentioned below and occasionally used as means of assistance.

The numerical simulation model for explosion load assessment requires consideration of likely sources and magnitudes of leaks, ignition and consequent explosion development. These are presently well addressed by CFD which is the most fundamentally based method and has the best potential for accurate prediction of gas explosion phenomenon. These tools solve the conservation equations of mass, momentum and energy including turbulence and combustion. A comparison of computational methods is presented by (Ledin, 2002) and includes extensive literature references and theoretical algorithms to describe gas explosions. The ventilation and dispersion analysis is needed to calculate the location and size of the gas cloud. The overpressure depends on the type of gas, the volume and concentration of the gas cloud, ignition source type and location, etc. The geometric details including process equipment, piping or vessels are modelled exactly as far as possible, because the explosion load is also affected by the degree of congestion and confinement of layout and geometrical shapes of the impacted structures. Therefore, the processing and building methods of 3D CAD data or CFD mesh for various objects and situations are also needed.

5.2.2 *Probabilistic approach*

The explosion accident is a complex phenomenon derived by a number of random variables, which have many uncertainties. Figure 9 shows steps in explosion modelling and intermediate results. The design random variables considered in calculating explosion loads are gas leakage rate and direction; wind speed and direction; locations of ignition; gas clouds size, location and concentration.
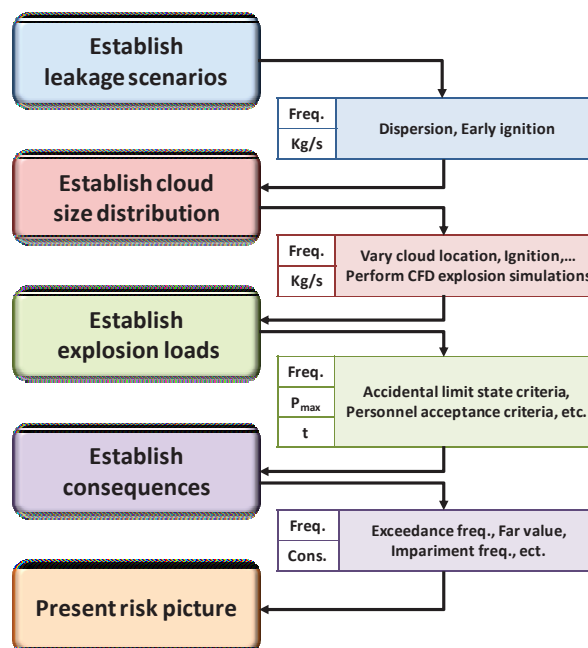

Figure 9. Schematics of procedure for calculation of explosion risk (NORSOK-Z-013, 2010).

The distribution of explosion loads on the target facilities is defined based on numerous explosion scenarios using sophisticated analysis methods, like CFD. However, the cost of simulating all of possible scenarios is very high. The probabilistic approaches in the present industrial practices were presented by Czujko (2001):

- Simulating a very high number of scenarios using simple models (e.g. PHAST/DNV),
- Performing a low number of CFD simulations and extrapolating the results based on previous project data
- Semi-probabilistic and risk-based approach implemented into an industry standard (NORSOK-Z-013, 2010) where accidental scenarios for gas explosion analysis are selected based on frequencies of parameters affecting scenarios. Resulting probabilistic models of explosion loads, i.e. exceedance curves, are derived from frequencies of scenarios implemented. This approach brings more accurate CFD calculations into risk analysis in a consistent and transparent way. The order of 300 ~ 400 CFD simulations is typically performed in this approach.
- Generic probabilistic approach where a selection of scenarios for explosion analysis is based on a probabilistic procedure, and a probabilistic model of explosion loads, i.e. exceedance surface, is developed based on numerical and probabilistic simulations.

While faster results are acquired in the first two approaches above, very conservative estimates for the risk as well as wrong trends with regard to sensitivity studies can be expected. The third approach, risk based probabilistic explosion loads assessment, is still under development with better understanding and modelling of the physics.

For the last approach, the overall concept (Czujko, 2001) and a few remarkable studies have recently been presented (Czujko and Paik, 2010, Paik and Czujko, 2012) However, this method has not yet been introduced in the industry and is under development, especially as far as the exceedance surface is concerned. The explosion loads are expressed not by a singular value but two components: explosion peak pressure and its duration (or impulse). In the case with two load-parameters, design values may be obtained from the joint probability distribution by contour curves. That is, the design load shall be established based on the probabilistic relation between the predicted explosion pressure and its duration from numerous scenarios (NORSOK-N-003, 2007, NORSOK-Z-013, 2010, LR, 2014c). Moreover, each load component has a great effect on structural safety. When one component of an explosion load is changed, the dynamic response of a structure under explosion loads shows inconsistent tendency (Biggs, 1964). Therefore, selection of the adequate duration corresponding to the defined pressure is an important issue in determination of design explosion loads.

In principle, the probabilistic explosion load distribution should be presented as a frequency distribution of overpressure and impulse, i.e., a P-I surface in a frequency space. Because of the two parameters (P and I) it is not possible to cut the P-I surface in a unique way such that a unique load-exceedance curve can be established as long as the design has not been finalised and the response characteristics of the structure are unknown (not decided upon). However, for a quasi-static or impulsive response case the P-I surface can be simplified to an overpressure-frequency or an impulse-frequency relation respectively. For the quasi-static case a simple overpressure exceedance curve can thus be established where the DAL pressure is determined by the risk acceptance (i.e., frequency cut-off) criterion. The associated range of impulse (or duration of overpressure pulse) for the DAL pressure can then be found from the scenarios representing the DAL pressure. For use in the impulsive or dynamic domain of the structural response the DAL can be presented as a triangular pressure pulse with a duration representative for the cut-off frequency. The duration can typically be defined by the 10 per cent overpressure points before and after the pressure maximum.

The explosion loads of all scenarios can be listed in descending order and the value whose cumulative exceedance frequency corresponds to risk acceptance criteria, i.e. 1.0×10^{-4}/year is selected as the dimensioning accidental load. However, in the present industrial practices for offshore explosion analyses, pressure and duration (or impulse) are considered separately and not treated as a combined term when design explosion loads are determined with given acceptance criteria (Figure 10).

Kim and Kim (2014) proposed a method to better integrate overpressure and impulse using an exceedance surface approach. The expected explosion response from a series of 8 random variables including wind and leakage data that determine the gas cloud, its concentration, position and size were used to combine overpressure and impulse (P-I) parameters into a joint probability density function and thereby provide a more realistic definition of the explosion load criteria (Figure 9). Applying the general risk acceptance criteria for the exceedance frequency of 1×10^{-4}/year allows developing sets of pressure and impulse values as a curve shown in Figure 10. Within the load limits by asymptotic points, the response frequency of the structure, T can be then used to determine one of design points rather than the greater loads suggested by the P-t and dual curve methods.
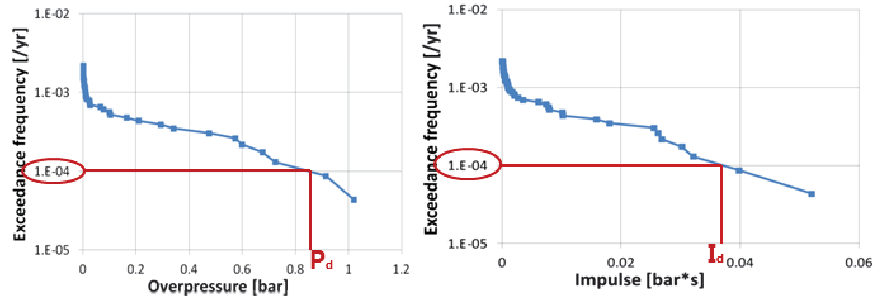
(a) Dual curve method
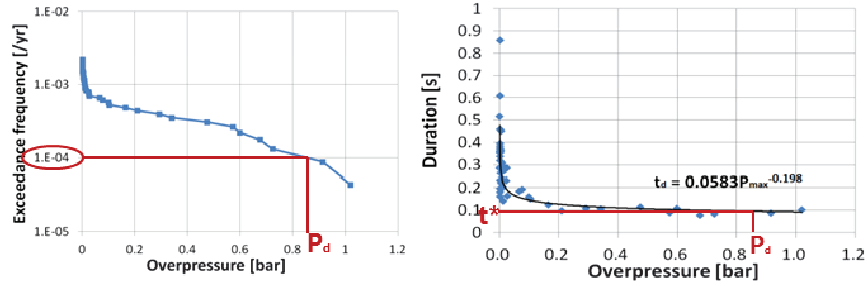
(b) P-t relationship curve method

Figure 10. Determination of dimensioning explosion load with exceedance curves (Kim and Kim, 2014).
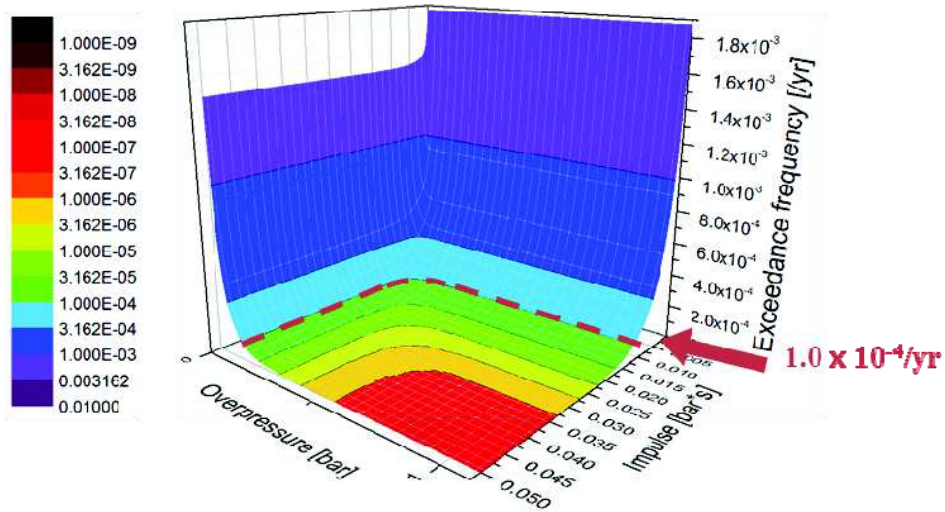

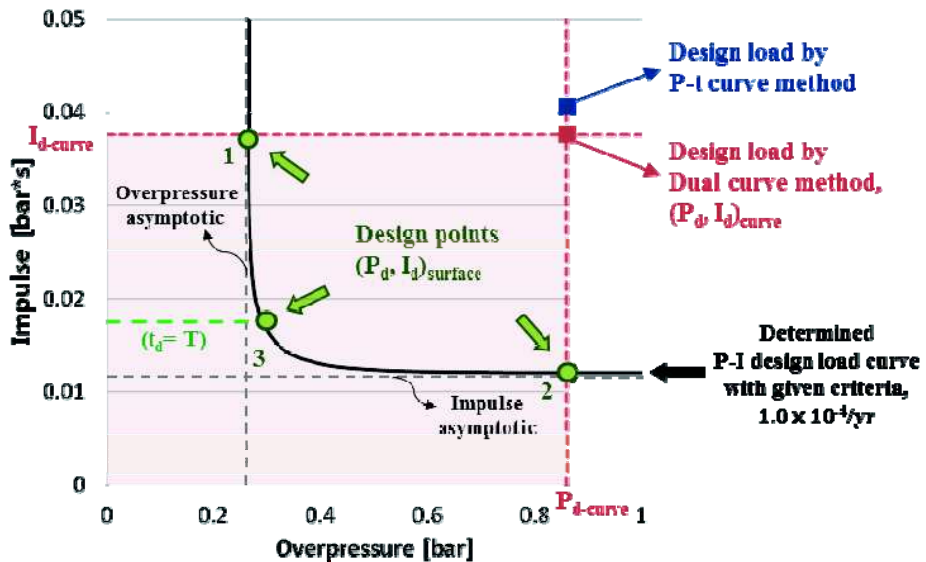Figure 11. Exceedance frequency surface for design explosion load (Kim and Kim, 2014).

Figure 12. P-I design load curve from exceedance surface (Kim and Kim, 2014).

5.2.3   Definition of explosion loads for design

The design explosion pressures for a topside structure calculated using CFD analyses are expressed as representative values for each area of the topside. However, depending on the size and shape of structures and equipment in the area, these explosion pressures generate different types of loads, that is, overpressure, drag pressure and differential pressure. Their detailed definition or implicated structural objects are well described in ISSC (2012). An example application of these loads is shown below, and the coefficients can be different in real projects.

- Blast load for Structural members (wall, deck, beam, column, panel)
- Overpressure and drag load
- Blast load for Piping
- Drag load, F_D = Dynamic pressure × Drag coefficient (C_d = 1.2) = ½ × ρ × A × v^2 × C_d

where, ρ is the fluid density, A is the maximum cross sectional area of the object in a plane normal to v, and v is the large scale fluid velocity ignoring spatial fluctuations in the vicinity of the object.

- or Drag load, F_D = Local overpressure × 1/3
- Blast load for Vessels
- Pressure differential load, F_P = ΔP × section area × PDF

where, PDF is pressure distribution factor(for cylinders, 2/π) and ΔP is the differential pressure across the obstacle.

- Blast load for Grating floor
- Drag load, F_G = Dynamic pressure × Drag coefficient (C_d =2.0) × Permeability factor
- Blast load for large items
- Reflection load, F_R = Overpressure × Reflection coefficients (2.0 for front wall)

5.3   Fire Load Assessment

The determination of the rational design loads for fire has the highest priority for coherent and reasonable evaluation of structural redundancy and life safety under serious thermal loads. A fire accident is also caused by combined effects of many random variables, which have high uncertainties and affect the incidence and progress of a fire accident. Therefore, the probabilistic approach is needed instead of the deterministic approach. The design fire load satisfying given acceptance risk criteria shall be determined in terms of heat flux and its corresponding duration because the structural response is highly influenced by both of the two load components. However, at present, there is no practical way to deal with two load components at once in the offshore fire design. Treating heat flux and duration independently could result in a mistaken prediction due to the omission of correlation existing between them.

5.3.1   Deterministic approach

Fires present a major hazard to offshore platforms. Typical heat load limits are reported by (Moan et al., 2002), NOSOK S-001 and FABIG to generate in the order of 200 to 300 kW/m^2 and last between 15 minutes and two hours. DNV-OS-A101 rules also define requirements for critical items in liquid and gas collection areas and specify that fire load requirements for structures which have critical items shall be to withstand a jet fire (250 kW/m^2) for 30 minutes and a pool fire (150 kW/m2) for the following 30 minutes. In areas with only oil or condensate containing equipment, critical items shall be designed to withstand a pool fire (150 kW/m^2) for 60 minutes. Areas with only gas containing equipment shall have critical items designed to withstand a jet fire (250 kW/m^2) for 30 minutes. Additional classical algorithm based fire spread modelling methods that determine probabilities, fire growth, temperature time histories and correlations for full room involvement in marine structures are reported by Sprague and Dolph (1996).

The nominal fire loads similar to explosion case were suggested in various documents (API, 2006, NORSOK-S-001, 2008, DNV-OS-A101, 2014, FABIG, 2014). In these documents, tabulated guidance for each fire type (pool fire & jet fire) are provided including their typical properties like flame size, heat flux, temperature, etc. In the case of numerical models for consequence of a fire load, PHAST software is widely used with 2D simplified model and KFX or FLACS software can be used with 3D CFD model.

5.3.2 *Risk-based and probabilistic approach*

The probabilistic fire load for offshore facilities is usually determined in the process of fire risk assessment. The scope of the fire risk analysis is to identify potential major fire hazards on the installation. The specific objectives are to:

- Identify the fire scenarios resulting from process or non-process failure cases
- Evaluate the frequency of fire scenarios
- Quantify the consequence of the identified scenarios (flame length, pool diameter, heat flux contours)
- Identify the targets vulnerable to the fire scenarios and evaluate their consequences
- Identify the escalation potential of each accidental event and the possible damage to the asset.
- Identify the possible requirement for additional risk reduction measures to help prevent and/or mitigate the effects of the identified fire scenarios (safety distance of the restricted Area, Passive Fire Protection, fire wall requirement, etc.)

The topside process segments are divided into isolatable segments bounded by Emergency Shutdown Valves (ESDV), and these are further broken down based on the operating conditions, compositions, phases and location etc. In the context of the fire risk analysis, the ESDVs and SDVs are crucial to ensure a safe isolation and shutdown of the process system in the event of an emergency (i.e. detection of hydrocarbon release or fire).

The leak failure frequency of each isolatable segment is based on historical data and derived by counting the associated equipment and fittings e.g. vessels, heat exchangers, valves, flanges, instruments and piping, etc. The fire frequency from a leak event is determined considering the probabilities of ignition, early detection, isolation and blowdown.

The consequence of the fire event can be modelled by 2D simplified method (e.g. PHAST) or 3D CFD method (e.g. KFX) and expressed with flame length, flame diameter, heat flux contour or impairment frequency, etc. These loads are assessed against design criteria. The design fire load for the target can be determined by comparing the fire loads with the likely fire resistance of the potential targets in order to assess the likely frequency of credible damage scenarios.

For probabilistic fire modelling, it is needed to establish dimensioning fire loads. The fire load have two parameters, that is, incident heat flux and duration, and have strong spacial and time dependency. One problem is that there is no unique and unambiguous way to derive dimensioning fire load considering these two load parameters ((NORSOK-N-003, 2007, LR, 2014b).

Recently, semi-probabilistic and risk based approach for fire design was suggested (FABIG, 2014) as the explosion case. In this procedure, 1×10^{-4}/year DAL dose and dimensing scenario are introduced from exceedance curve of heat dose. however, this heat dose must be converted back to a flux and duration, and therefore, there is still some weakness that the time history profile of the fire load or correlation between heat flux and duration is not exactly expressed. Huser and Vianna (2010) and Huser (2013) outlined a probabilistic procedure for fire loads based on the same principles used by NORSOK-Z-013 (2010) for explosion analysis, and also explained the dimensioning heat dose application of FABIG (2014) as shown in Figure 12. Paik and Czujko (2012) showed the probabilistic procedure for fire loads with exceedance curves of both temperature and heat dose (Figure 13). Kim and Kim (2015) propose a method to better integrate heat flux and duration with maintaing the time history shape of fire loads by using joint probability distributions and exceedance surfaces among heat flux, heat dose and leak duration similar to their previous work(Kim and Kim, 2014) for explosion loads (Figure 13).
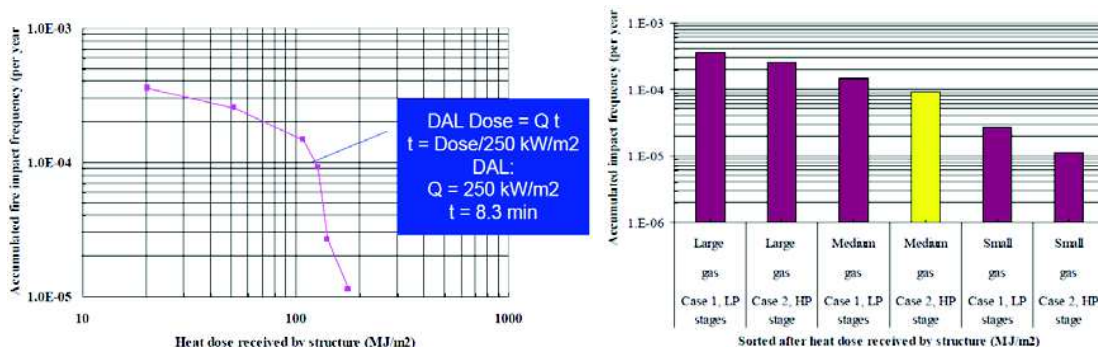

Figure 13. The exceedance curve of heat dose and DAL fire scenario Huser (2013).
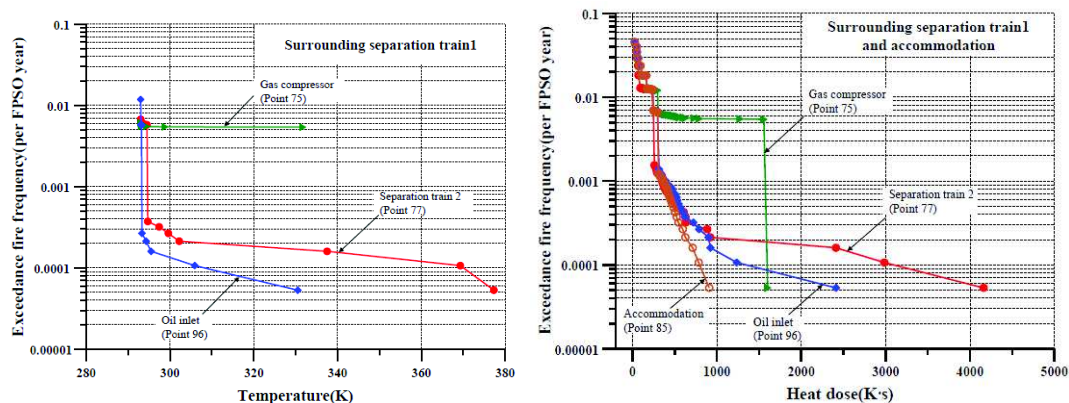
Figure 14. The exceedance curve of temperature and heat dose (Paik and Czujko, 2012).
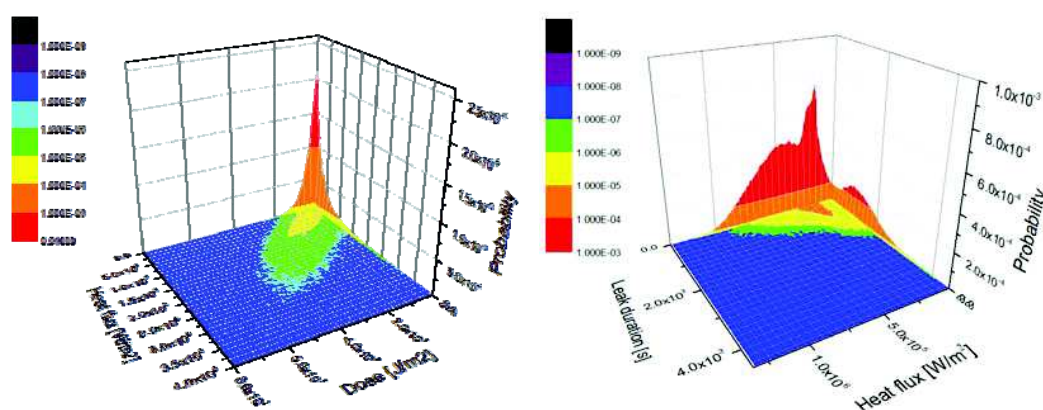

Figure 15. The joint probability distribution for heat flux, heat dose and leak duration (Kim and Kim, 2015).

5.4   *Load Assessment for Collision accidents*

A ship collision with an offshore structure takes place when a support, off-load or trading vessel lose their station keeping ability for various reasons and collide with the offshore facility. Ensuing damage to both structure and ship include large deformation of primary structures as well as cracked joints, tears in plating and ruptures of various tanks or other stability control systems. The preliminary assessment defines the acceptance criteria, the collision events, and collision risk. The detailed assessment selects more specific pertinent preliminary scenarios and identifies mitigation measures to meet the criteria and re-assess the event using less conservative analysis techniques to evaluate the local and global expected performance and re-evaluate the risk to determine whether additional mitigation is required.

Departing from the conventional structural design concept where functional loads are mainly concerned, considering the accidental loads in design phases is getting increased to reflect its serious impact to determine the required structural integrity. Collision accidents are one of the important potential hazards, to which ships and offshore structures are easily exposed. Therefore, the assessment of their consequences to ultimately minimize or prevent potential damage should be performed and it is preceded by defining design collision load. Generally, the collision design load is determined by deterministic or probabilistic approach.

One of the practical ways to determine design collision loads is using design codes produced by the various regulatory authorities or classification societies. However, no standards are universally accepted and requirements of specification are different for each offshore project. Therefore, procedures to determine design loads should be properly selected project by project. Another deterministic approach is based on a few scenarios, which are the worst-case type or the cases of some typical collision accidents. In this way, analysis method of external mechanics is employed to estimate the amount of kinetic energy available to be dissipated to deformation energy during collision.

Over the past decades, risk assessment methodologies for ship and offshore collision problems have increasingly been applied and more realistic design collision loads have been determined reducing the conservatism of the deterministic approach. In this approach, the frequency and consequence of each collision scenarios are determined. The frequency of the collision is related to the visiting or passing frequency of vessels which sail or operate near the field where the installation is located and the

probabilities of colliding conditions (i.e. approaching angle and speed, wave and wind conditions, etc.). The consequence of each accident is determined in terms of impact energy by the external mechanics. The results of the risk based approach can be explained by employing the risk matrix whose row and column represent the collision frequency and consequence respectively, or the exceedance frequency curve of collision impact energy.

5.4.1 Deterministic approach

The process involved in collision is very complicated and many predictive calculation procedures have been developed for predicting collision impact energy and its response. Collision mechanics can be classified into two parts, namely external mechanics and internal mechanics (ISSC, 2006, ISSC, 2012). Those two parts are often treated separately, but in some cases, they are solved together.

The first point of reference for the designer is the codes of practice produced by the various Regulatory Authorities and/or Classification Societies. The codes for collision accidents follow approximately the same approach that is the assessment of impact energy to be absorbed by the struck objects. Collision with 5,000–7,000 ton supply boats with speed of 0.5–2 m/s has been investigated as the most probabilistic event. Thus, the codes are explicit with this type of collision. A tabulated summary of the codes was given in the HSE report by MSL-Engineering (2000), and the overview of existing guidance on collision assessment is introduced in LR (2014a). In practical offshore projects, project's specification states the design value and acceptance criteria in detail and these are based on the experience of previously-built installations, relevant rules, regulations and the results of engineering research.

5.4.2 Risk-based and probabilistic approach

The objectives of collision risk analysis are to identify potential ship collision scenarios, evaluate the likelihood of their occurrence, determine their consequences and impact energy, and to assess the risk of ship collision. Therefore, design accidental loads for collision can be selected by assessing the impact energy against the design acceptance criteria for risk or performance. The LR guidance notes for collision analysis (LR, 2014a) provide an overview of the methods and standards to perform risk based assessments of collision events.

Collision scenarios define the typical collision events and their probabilities to be considered in design. Parameters needed for defining collision scenarios include the type and geometry of a colliding vessel, colliding speeds, colliding angles, collision locations, cargo loading conditions, drafts and so on.

Colliding ships may be categorized into passing vessels (not related to the installation like shipping traffic including merchant vessels, fishing vessels, passenger vessels, naval vessels, etc.) and visiting vessels (to serve the installation like supply vessels, export tankers, and product tankers, etc.). Installation dimensions and orientation, shipping data (traffic density, type and size), visibility, wave height, vessel speed distribution, collision risk management system, i.e., standby vessel, etc. are factors influencing collision frequency of passing vessels. The collision risks associated with visiting vessels are different from passing vessel collisions, as they occur more frequently, and usually have lower energy impacts. However, the relatively higher frequency of these low energy impacts can increase the cost of repairs over the lifetime of the FPSO to a substantial level. It is also possible to differentiate ship collisions according to whether or not the colliding vessel is under power at the time of the collision, i.e., Powered or Drifting.

Impact energy calculations for passing and visiting vessels are used to assess vulnerable areas of the installation. The total impact energy of a colliding vessel with a static installation is equal to the kinetic energy of the vessel immediately prior to impact, i.e. $E = \frac{1}{2}mv^2$, where E = colliding kinetic energy; m = the effective mass of the colliding vessel, $m = m_s + dm_s$; v = the velocity of the colliding vessel immediately prior to impact; m_s = displacement of the colliding vessel; dm_s = added mass of the colliding vessel. For the rigorous expression refer to Pedersen and Zhang (1998). The 'added mass' term is a factor that takes account of the hydrodynamic forces that act on a vessel during a collision. This factor depends upon a considerable number of parameters but the simple approximations are commonly adopted: $dm_s = 0.1m_s$ for a bow/stern on collision, $dm_s = 0.4m_s$ for a side-on collision. Some of this kinetic energy will be absorbed in elastic/plastic deformation of the colliding ship's structure and some will be absorbed in elastic/plastic deformation of the structure. For glancing blows, the colliding ship may also have a residual kinetic energy after the collision.

The impact energies of all scenarios can be listed in descending order and the value of which cumulative exceedance frequency corresponds to risk acceptance criteria, i.e. 1.0×10^{-4}/year is selected as the dimensioning accidental load. This dimensioning load is assessed against design criteria. The design impact load for the target can be determined by comparing the impact energies with the likely impact resistance of the potential targets in order to assess the likely frequency of credible damage scenarios.

Youssef et al. (2014a) outlined methods to address hazard identification and probabilistic scenario selection for ship collision accidents. Collision scenarios are then described using a set of parameters that

are treated individually as random variables and analyzed by statistical methods to define the ranges and variability to formulate the probability density distribution for each scenario. As the consideration of all scenarios would not be practical, a sampling technique is applied to select a certain number of prospective collision scenarios. Applied examples for different types of vessels are presented to demonstrate the applicability of the method. Youssef et al. (2014b) also reported a probabilistic risk assessement for ship-ship collisions, in which not only the total collsion risk but also the exceedance curves are established that can be used to define the collision design loads in association with various design criteria (Figure 16).
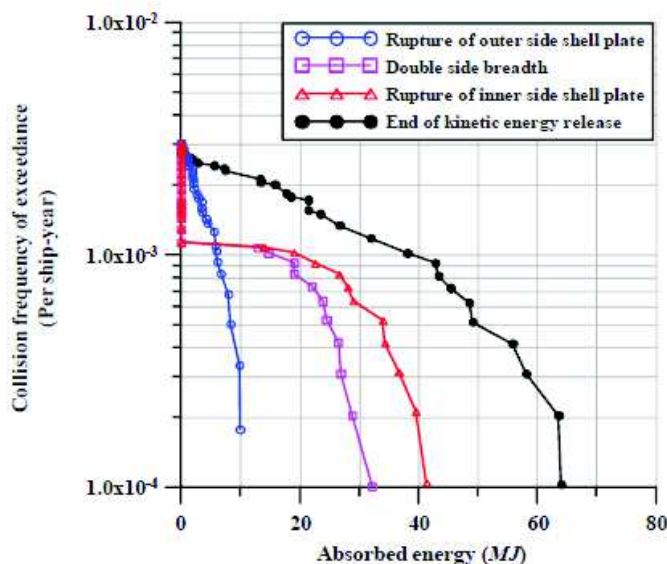

Figure 16. Collision exceedance curve in terms of exceedance of collision frequency versus absorbed energy
          (Youssef et al., 2014b).

5.5   *Load Assessment for Dropped Object accidents*

The dropped events generally arise during the transfer of cargoes by cranes, and these are rarely critical to the global integrity of the installation and will typically cause local damage. However, these events may bring about serious consequences such as the personnel fatalities or the system shutdowns, so it is important to evaluate the structural strength to prevent these losses.

   The process for assessing accidental loadings from dropped objects follows in principle the risk assessment traditionally used by the offshore industry, in which expected dropped events are developed with their likelihood and then critical scenarios and design impact energy is decided considering the consequences of those events. The impact energy is characterized by its kinetic energy determined by its mass including hydrodynamic added mass if it falls through water and its velocity. The ALS design is performed for this impact energy and has to satisfy the design requirements. The kinetic energy has to be dissipated as strain energy in the impacted component and possibly also in the dropped object itself. Generally, this involves large plastic strains and significant damage to the impacted component, which shall comply with ductility and stability requirements.

5.5.1   *Deterministic approach*

It's hard to find the specific references for the impact energy of dropped objects. Rules or standards show only general requirements and can be summarized with two criteria: no penetration of impacted deck structure and no safety critical system's shutdown due to large deformation. For example, the class rule (BV, 2014) requires that the energy of the dropping object at the moment of the contact with the deck is to be lower than the absorbed energy by the deck deformation at the limit state. The limit state corresponds to the first rupture of a plate in the deformed deck area. NORSOK-N-004 (2004) states that the masses of the dropped objects are normally based on operational hook actions of the platform crane and critical areas of dropped objects are to be determined based on crane operation sectors, crane reach and actual movement of actions. DNV-OS-A101 (2014) advises that the weights of the dropped objects to be considered for design of the structure are normally taken as the operational hook loads in cranes. The impact energy at sea level is normally not to be taken less than 5 MJ for cranes with maximum capacity more than 30 tonnes. The impact energy below sea level is assumed to be equal to the energy at sea level. In practical offshore projects, their specification sometimes states the minimum impact energy (usually

hundreds kJ) and acceptance criteria and these are based on the experience of previously-built installations, relevant rules, regulations and the results of engineering research.

5.5.2   *Risk-based approach*

The purpose of dropped object risk analysis is to assess potential dropped object scenarios with frequency and their consequences. The dropped object risk analysis includes:

- A comprehensive register of all lifts including the type of object, size, weight, frequency, etc.
- Identification of the hazardous areas and safety critical elements susceptible to be damaged due to dropped object from mechanical handling activities
- To assess the likely frequency of dropped objects onto potentially vulnerable targets, both on the installation and overboard/subsea
- Estimation of the frequency of dropped objects onto potentially vulnerable targets on the installation or overboard
- Assessment of the impact energies and of the effects of such impacts
- Evaluation of the results against the risk acceptance criteria; when necessary proposition of mitigating actions, including restricted areas for normal crane operations, to reduce the risks to as low as reasonably practicable (ALARP).

Frequencies of dropped objects can be taken from the failure databases (OREDA, 2009, OGP-434-8, 2010). The subsea analysis is performed usually in accordance to the methodology provided in the DNV-RP-F107 (2010). The impact energy is calculated according to the kinetic energy of the dropped object, i.e. $E_k = mgh$, where E_k = Kinetic energy; g = Gravitation acceleration; h = Height of the dropped object. The impact energies of all scenarios can be listed in descending order and the value of which cumulative exceedance frequency corresponds to risk acceptance criteria, i.e. 1.0×10^{-4}/year is selected as the dimensioning accidental load. This dimensioning load is assessed against design criteria. Figure 17 shows the typical exceedance probability distribution of impact energy for the dropped objects.
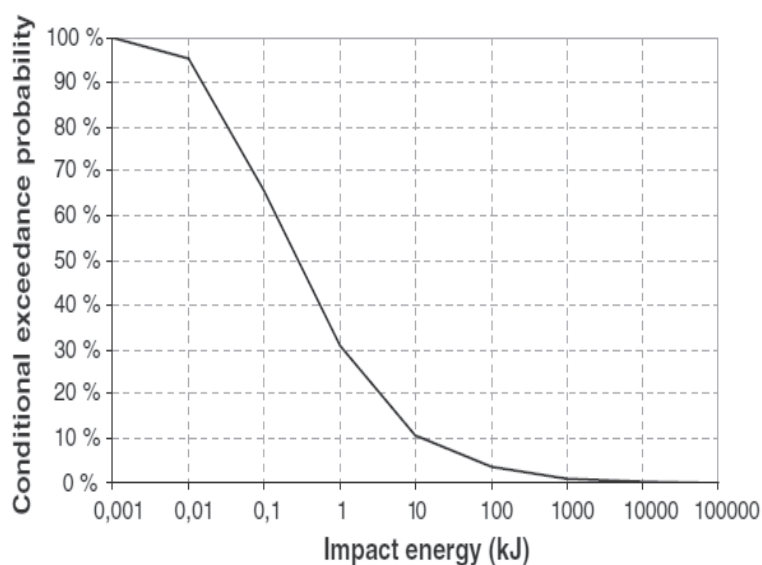
Figure 17. Impact energy distribution of dropped objects on production installations, 2002~2001, NCS (Vinnem, 2014).

6.   DETERMINATION OF ACTION EFFECTS

6.1   *Introduction*

The determination of action effects is the next step in ALS process as a continuation after identifying accidental actions. Actions are necessary to improve the design and further develop design standards and safety limits. A successful determination of action effects is therefore necessary for the evaluation of structural integrity either as a direct consequence of accidental actions, or as an escalation of the initial event; the decision if the structure is compliant can then be made. A proper evaluation should include the

appropriate analytical model or software capable of capturing the action effects, and their non-linear attributes. The determination of action effects may lead to the definition of appropriate metrics, or the adoption of existing limiting criteria.

For the evaluation of action effects the analysed structure has to be represented with a mathematical model including the description of geometry, material and loads. Depending on the problem this mathematical model can be a closed form analytical equation, simplified beam model based on rigid-plastic analysis, or a large numerical model consisting of shell, beam and solid elements suitable for non-linear analysis. Simplified models often have limited usage and accuracy making them suitable for studies where a large number of analyses are to be conducted. Typical examples, where such simplified models are used are risk analysis studies for ship collision in a certain sea region, see for example Goerlandt et al. (2012), Montewka et al. (2014) or Qu et al. (2011). Goerlandt et al. (2012) were applying simplified consequence models to study the probability of oil outflow in tanker collisions in the Gulf of Finland. They concluded that using more accurate collision energy and/or structural capacity models for purposes of maritime traffic risk analysis is currently not worthwhile, as the uncertainty regarding impact scenario is overwhelming- thus indicating the sufficiency of simplified tools. Another model focusing on the simplified methods and collisions in the Gulf of Finland by Montewka et al. (2014) presents a probabilistic model for collision accidents and successfully validates it with historical data. Qu et al. (2011) studied the collision probabilities in the Singapore Strait and proposed a series of measures, such as speed limits and passage guidelines, to reduce collision risks. Simplified consequence models for collision can be based on the deformation mechanisms or on numerical simulations. Haris and Amdahl (2013) and Gao et al. (2014) are examples of simplified models that sum the crushing resistance of different structural components to form the collision response of the whole side structure. By combining numerical and analytical approaches, Ehlers and Tabri (2012) presented a method that calibrates a semi-analytical collision model based on single numerical simulation allowing for a rapid assessment of different collision scenarios for a given ship.

Recent work on the subject of nonlinear structural response analysis of offshore installations under explosive loads, with coupled load mapping between CFD solvers and nonlinear structural analysis is presented in Paik et al. (2014). This work shows that rule-based uniform load application can result in an overestimation of structural damage, and in other cases it may lead to an underestimation. The traditional SDOF approach is also known for returning different nonlinear structural responses depending on the blast load profiles (usually triangular load for dynamic structural analysis when considering hydrocarbon explosion accidents). Simplification of the topside structure when assessing blast effects often overlooks the uncertainties associated with load profile idealisations (triangular, rectangular, symmetric triangular, or linearly decaying).

Dropped objects analysis is closely related to collision analysis following general guidance notes for impact actions. Impact scenarios can be identified from risk assessment considering crane lifting capacity, crane reach, and laydown area capacities, where lift frequencies and mass of lifted object are considered. Acceptance criteria take several forms not only based on Class Rules but also on the operator's operational requirements. BV (2014) recommend safety principles dictating that structural elements may suffer permanent deformations without any rupture. NORSOK N-004 Annexes A.4, K4.4.4, L.8.1 and M.6.2 provide guidance with simplified methods for resistance/energy dissipation expressions and selection of critical areas of dropped object analysis.

Requirements from the operator's point of view may be specific in terms of impact energy on specific areas on deck or equipment, particular laydown capacity on deck plating, or even deck plating designed to be capable of withstanding explicit impact energy based on a standard ISO container from a specific height with the assumption that a percentage of the overall impact energy is to be transferred to the structure through elastoplastic deformation.

If no suitable simplified model is available or when a specific structure has to be studied at higher detail, numerical models can be used. Numerical models are more suitable for treating material and geometrical non-linearity, highly dynamic loads and the coupling between loads and structural response. A numerical model aimed at establishing safety levels should be in appropriate balance between precision and time efficiency, yet results should be benchmarked against established, well-documented and accepted modelling and industry criteria. Academic studies usually provide more precise models that can be used to prepare benchmarks for the validation of software and modelling approaches. The challenge is in finding the right balance between sensible analysis time and results that couple continuous design updates of hazard analysis with design iterations.

Several challenges in the determination, analysis and evaluation of action effects using numerical models are discussed in this section with emphasis on uncertainties associated with ALS conditions.

6.2 Review of Numerical Tools

The structural consequences of accidental loads, as identified in Section 2, may be assessed in various ways; by energy considerations combined with simple elastic-plastic methods (DNV-RP-C204, 2010), or by coupled advanced numerical methods. Accidental loads, and ultimately their possible escalation into an accident disproportional to the initiating event can be addressed in the design stage as shown in Annex 3. This section is focused on the most popular numerical tools used nowadays.

Software often used is based on linear elastic analysis, and it extends in the non-linear domain with advanced strength and failure models. The finite element method is the most popular among structural analyses when accidental loads are investigated, while more advanced analyses may include fluid-structure-interaction solutions, gas dispersion models and thermal effects. Despite advanced models, closed-form solutions are preferred where applicable, due to their simplified use, and the avoidance of convergence setbacks.

From modelling point of view the challenge is not in using numerical methods but rather using the available tools appropriately. A balance between structural detail and confidence in results should characterise a model.

In ALS design where non-linear action effects are dominant, FE analysis is practically axiomatic. Implicit and explicit algorithms can be used with the two terms referring to the time integration schemes. The former is solving for displacements at the nodes through the inversion of the stiffness matrix, and applied to the nodal out-of-balance forces; while the latter solves for nodal accelerations by dividing summation of forces by nodal mass. The specifics of time integration schemes can be found in the documentation of the FE code. Several limitations apply to both schemes but explicit solvers are relevant to short duration, high pressure gradient and large strain applications; often many times above the material's ultimate capacity. The experienced strain rate witnessed by the material determines the appropriateness of the solver. Table 2 is an unofficial guide showing the transition from implicit to explicit as strain rate levels increase.

Table 2. Solver Applicability per Hazard.

Solution	Impact Vel (m/s)	Strain Rate (1/s)	Effect	Hazard/Analysis
Implicit		$<10^{-5}$	Static/Creep	Fire/ Push Over
	<50	10^{-5} - 10^{-1}	Elastic	Ship collision
Explicit	>50	10^{-1} - 10^{1}	Elastic-Plastic	Collision, dropped object

Due to the large degree of non-linearity involved in ship collision analyses, implicit solvers are to be avoided as significant time is needed to achieve convergence. Explicit solvers may be used but the analysis time can be demanding with extensive modelling effort needed in the underlying assumptions and preparation (stabilisation energy and non-linear parameters often needed). Simplified expressions and industry-specific software that predict the collapse load of a member after a collision and the joint capacity are often used avoiding the intricacies of FE codes that can be demanding to master. Non-linear solvers like USFOS, Ls-Dyna and Abaqus are preferred for these problems where the loss of stiffness at large displacements and member collapse is considered. The output of concern is usually a force-deflection, or energy dissipation-deflection curve. Most effort should be spent on establishing credible failure criteria and creating a mesh size based on recommended practices where applicable. Most commercial codes nowadays support a series of failure criteria that remove elements as soon as certain limits are exceeded. Typical failure criteria are based on geometric strain, plastic strain, element distortion, or time step.

Gas dispersion and fire simulation are often analyses that operators undertake in the context of fire safety design to judge the integrity of the structure. Additional need for fire response analysis other than design purposes can be a result for accident investigation, part of risk analysis, or the need to study escalating effects of fires; often explosions. Heat loads from fires and structural fire response analysis in commercial codes use the k-epsilon model for turbulence modelling, the eddy dissipation concept for combustion modelling and a radiation model based on the discrete transfer method. Typically, the loads are presented as time-histories of ambient temperature around the structure or the heat flux acting at the structure. These thermal loads are applied to the structure either via using directly the temperature as a boundary condition, via heat flux boundary condition or using radiation and convection with prescribed ambient temperature history. Thermal problems can be solved with implicit solvers in the time domain. For steel structures under thermal loads it is the deformation of the heated members of primary concern, and not so much the ultimate strength as long as the maximum temperature remains below 400ºC. If the temperatures increase further, the ultimate strength and the

load carrying capability of the structure becomes of interest. Explicit solvers can be used for thermal analyses as well, but this approach can be computationally expensive. Implicit solvers are other preferable for collapse analysis under the thermal loads as the simulations tend to have long durations and the dynamic effects are neglectful. Thermal solvers implemented in Abaqus and USFOS are among the most popular tools used in the industry to assess thermal effects. A relatively new thermal solver in the market is FDS with the advantage of being free of charge as of 2014. The main drawback of FDS is that it is developed for low momentum applications of domestic fires and there is need for user interaction to use it for oil and gas applications. Finally, FLACS and FLACS Fire are well established tools for fire propagation and fire load assessment that have been used extensively over the years. DNV-GL Phast is another tool for hazard analysis for several stages until the action is fully established. Table 3. summarises the software options per accidental load.

Table 3. Software applicability per accidental load.

Load Type	Analysis Type	Solver	Software
Ship Collision	Non-Linear	Implicit/Explicit	USFOS, Ls-Dyna, Abaqus
Dropped Object	Non-Linear	Explicit, Hand Calcs	Ls-Dyna, Autodyn, Abaqus
Fire (thermal)	Linear/Non-Linear	Implicit/Explicit	FDS, FLACS, CFX, USFOS, FAHTS
Explosion & Dispersion	Non-Linear	Explicit, Hand Calcs	Ls-Dyna, Autodyn, FLACS, AutoReaGas

Anyone involved in engineering analysis is aware that numerical tools have a theoretical basis and are formulated to conform to experimental data. The failure mode in a ship collision or dropped object analysis should be of the expected type based on experiment observation, experience, and engineering judgement. The challenge in using advanced numerical methods is not so much in getting results, but rather to present an estimate based on valid engineering decisions made upon modelling. The challenge lies with the analyst in deciding if the simulated solution is representing a realistic response. Built-in features of non-linear solvers are constantly developing, and they are by no means free of defects. One should be cognisant of the underlying principles and assumptions made in the software. Today's software will implement "by default" settings requiring minimal necessity for user-input values; a simple model should be subject to a well-defined load/deflection curve in order to calibrate the results against any underlying models. Choosing the appropriate material model based on large deflections with a given rupture strain, is only an indication as guidelines and recommendations are not considering the mesh size of the model. A sensitivity analysis should follow the first solution to make sure that the output value is converging.

6.3   *Modelling Geometries*

Any structure needs to be modelled with sufficient detail so that a satisfactory response is captured when loads are applied, while the degree of detail depends on the type of analysis, as shown in Table 4. Thus, the detail level of the model depends on the scope of the analysis, the load character and the anticipated response. While the analysis under operational loads has to capture linear stress distribution, accidental and ultimate limit state analysis has to capture geometrical and material non-linearity with a finer mesh resolution at stress concentration areas. Some general guidelines for modelling can be found in DNV-RP-C208 (2013) and DNV (1999).

Furthermore, the action orientation and application method can sometimes determine the way a model needs to be meshed. Explicit analyses for example are less tolerant and highly sensitive to distorted elements (CFL limit), while implicit analyses can proceed with an inferior mesh quality. Advanced mesh algorithms are nowadays available, but certain fundamental criteria need to be met when meshing. It cannot be stressed enough that when trying to capture non-linear effects in implicit analyses, or in cases where action effects require the implementation of explicit solvers, reusing the implicit mesh is grossly erroneous. Engineers should also be aware that the implicit mesh is not suitable to capture the dynamic phenomena expected during high strain rate phenomena such as thinning, hardening, or thermal softening; the model should be re-meshed when working with inherited or legacy models. Internal checks are carried out where comments and warnings regarding the mesh quality are registered in the .out file of most codes. Messages contained in the .out file should be checked even at the end of a successful analysis. The higher the degree of detail in a model, the higher the likelihood to violate mesh criteria as a higher number of elements and nodes is

created. Typical mesh quality criteria are presented in Table 4 although a deviation from the values shown may be possible across FE codes.

Table 4. Element quality criteria.

Quality	Criterion	Description
Warpage	Less than 15deg	Deviation from planar plane
Aspect Ratio	Less than 5:1	Ratio of the longest edge to shortest one
Skew	Less than 60deg	Angle between two lines joining opposite mid-sides of the element
Jacobian	Greater than 0.6	Measure of the deviation of an element from an ideally shaped element

The type of accidental effect being studied greatly determines the degree of detail needed and the time spent to ensure that the mesh is satisfactory for the intended analysis, this is given epigrammatically in Table 4.

Table 5. Geometry Discretisation.

Item/Load Type	Operational Loads	Fatigue Loads	Accidental Loads	Ult Limit State
Analysis Type	Linear	Linear/Hot Spot	Non-Linear	Non-Linear
Mesh Size	Coarse	Very fine in SC areas	Fine to allow for buckling, tripping	Fine to capture local failure
Characteristic Mesh Size	Single element between stiffeners	$t \times t$ in SC areas	4-6 elements between stiffeners	4-6 elements between stiffeners

Most commercial FE codes support a direct import from CAD packages but industry-specific software is often limited to few geometry formats that limit the options when facing problems upon import. Removing any unnecessary details, unlikely to contribute to the stiffness of the model, is recommended not only for successful imports but also for avoiding singularities in the contour plot. In cases where CAD models consist of solids, the FE analysis may only require a surface model. Thus, a middle surface application is required. The benefit from working with shell instead of solid elements is the convenient way in which thickness is defined. Any thickness changes can be easily accommodated without the need to remodel. Often the non-linear model is developed from a model used for linear analysis. Linear model should be complemented with non-linear material properties and often the mesh resolution should be increased in the areas where non-linear deformations and effects are assumed to occur i.e. the collision zone or the drop zone for dropped objects analysis. Furthermore, in linear models the stiffeners are often replaced with beam models although this approach cannot consider buckling or tripping of the stiffeners. A shell-beam combination to model a stiffeners, or girders, is also possible where girder webs are modelled by means of shell elements and flanges may be modelled using beam elements.

A discretised model should describe the three dimensional geometries of the structure so that the non-linear solver can yield reliable results based on a geometry that can respond to the expected behaviour and allow for the anticipated failure modes to develop. Common failure types of structures include tension yielding, bending/compression failure, stability failure (the main failure mode in jacket structures), post-buckling load shedding in neighbour members, and interaction with local buckling. The implemented elements should capture these failure modes at all times. Yield and fracture modelling requires careful consideration on element size in order to capture high stress concentrations in the facture zone. Typically, several elements should be present in the yield zone to achieve good strain estimates (DNV-RP-C208, 2013). However, this discretisation level is hard to achieve with large structures and material failure criteria should be calibrated for larger element size as discussed in Section 6.5.3. Detailed FE models used for collision analysis can be seen for example in Storheim and Amdahl (2014) and Gao et al. (2014).

In the context of modelling for accidental effects the degree of detail varies. Beam models are by no means as detailed as those required in fatigue or stress studies where inclusion of joints and stiffeners is necessary. Beam members are often modelled with centre-to-centre length properties for conservative and simplicity reasons. Modelling with face-to-face properties reduces the buckling length, thus increasing the member's capacity. Hellan (1995) reported that the effect of face-to-face modelling can increase the capacity by 5%. The number of beam elements per span should be carefully selected as excessive number of elements can return spurious plastic hinges particularly in non-linear analyses. An exception to the above is modelling for buckling response where multiple beam elements, usually 4 to 6, per span are used. In short, the structural members needed are the ones that provide stiffness and strength.

Stiffened panels can be fully modelled or simplified by using orthotropic modelling techniques, or the combined stiffener method. Asymmetric stiffeners, like HP profiles, are often modelled as flat bars with equivalent properties. Any offsets are to be addressed by flushed stiffeners or beams and this is treated as shown in Table 5 based on the studied hazard.

Secondary members consist of members that do not contribute to the global stiffness or strength of the structure. Should secondary members attract any loads, these should be modelled with sufficient detail so that the loads are transferred onto the primary structure realistically. A typical example of this is the conductor framing and the boat landing fenders. Any inclusion of secondary members in the global model should be justified as they can affect the results with excessive deformations (in the load-deflection curve) when heavily loaded. Members that contribute to the overall capacity of the structure should be modelled. Members not included in the model can attract hydrodynamic forces and additional loading due to their own weight. In practice, omission of secondary members is usually accounted for by factoring up the material density but this technique may not be always applicable.

Table 6 gives the modelling requirements per hazard considered in this section as the degree of detail and structural arrangement varies. Particularly in advanced analyses where CFD analysis precedes the structural assessment, the layout of the deck area needs to be modelled accurately to allow the formation of poor fire if that is possible. In most cases a beam/stiffened panel assembly is simplified to a beam-only model. In the full structural configuration the stiffened panel may, or may not, have its stiffeners or beams flushed to plate level.

Another detail pertinent to beam-only models, often used in thermal analyses, is the applied boundary conditions. In a fully fixed condition the beam node is fixed in all 6 degrees of freedom, while in a shell model the upper, lower flange and web nodes can be fixed if necessary. Shell elements are not rare as all surfaces (flanges and web) in the model can be considered to be engulfed (worst situation) at the same degree of heat flux. Another benefit is that radiation among neighbour surfaces can also be considered in a shell model. The benchmark study of this Committee showed that in heat flux application USFOS and LS-Dyna agree very well.

The application of passive fire protection (PFP) is not explicitly modelled but rather considered through the effective heat transfer coefficient of U-value in advanced fire analysis.

Table 6. Modelling Requirements.

Action	Structural Detail	
Collision	Plated structure with flushed members	
Dropped Object	Beam/plated structure with flushed members	
Blast	Plated structure with flushed members, stools, structural foundations	
Fire/Thermal	Plated structure or beam-only structure included. No flushed members	with, or without, PFP

6.4   Modelling Loads

The way loads are applied can greatly alter the structural response of a structure. ALS analysis implies that the actions are of dynamic and non-linear nature, which could significantly affect the initial structural configuration and topology.

Considering the nature of accidental loads such loads are characterised by the following aspects:

a. Kinetic energy governed by the mass of the striking body (ship including added mass, dropped object, projectile mass)
b. Impact energy of the striking body, and the struck body if necessary
c. Impulse of overpressure
d. Temperature rise and duration of thermal effects

The following sections discuss modelling loads per hazard in detail.

6.4.1   Ship Collision

Ship collision analysis is often conducted in displacement controlled manner i.e. the analysis of external motion dynamics is decoupled from the evaluation of the structural consequences. Such analysis assumes a certain prescribed penetration path. It has been shown that such assumption is valid for symmetric right angle collisions while in non-symmetric collisions the penetration depth depends on the ship structural configuration, masses, collision location etc. Differences in damage description and penetration depth can

be significant. Brown (2002) compared the coupled model SIMCOL to the decoupled model of Pedersen and Zhang (1998) and concluded that, while the total energy is similar in both approaches, the decomposition of the total energy into transverse and longitudinal energy differs significantly. The same conclusion is drawn by Tabri and Broekhuijsen (2011) using coupled and decoupled finite element simulation to conclude that the decoupled simulation can result in erroneous description of penetration depth in oblique angle collisions.

Regarding ship collisions, the prescriptive scenarios exist for ship-platform or ship-FPSO collisions, see for example recent guidelines by LR (2014a). In offshore ship collision studies in design phase the speed of the striking vessel is not to be less than 2.0m/s while the most probable impact location should be determined by risk analysis with due account of the factors that affect the exact location like tidal changes, and vessel motions sea states (NORSOK-N-003, 2007). In the absence of specific information about the impact zone, values between 10m below LAT and 13m above HAT ((NORSOK-N-003, 2007) are to be considered. Force-indentation, energy-deformation and force-deformation curves are available in DNV-RP-C204 (2010) for different ship sizes and impact location (bow, stern, broad side). Should a more detailed investigation be required than the guidelines provided in classification rules, then an local explicit FE analysis should be considered with emphasis on material properties and failure criteria.

Such prescriptions typically do not exist for ship-ship collision. The accidental scenarios are defined from a risk analysis considering a limited sea region (Qu et al., 2011, Goerlandt et al., 2012, Montewka et al., 2014) or considering neighbouring ships sailing in the region, (UN, 2013).

6.4.2   Dropped Objects

Due to the short duration event, dropped object analyses are usually solved with explicit solvers where the kinetic energy of the dropped object converted to strain energy in the struck and the striking body is captured. Material failure is also considered in these studies, where implicit element formulations would be inadequate to accommodate. These models are often local with sufficient detail as shown in structural drawings. Special attention should be paid to the enforced boundary conditions; fixed conditions to the struck body often applied at a distance away from the striking point are typical. The striking body can be modelled wholly or partially. In the latter case the initial velocity corresponding to the whole body should be applied. Certain codes like AUTODYN can only return pressure results when using just solid elements, while Ls-Dyna can output pressure contours with shell elements as well. This subtle difference can have consequences for both modelling and computational effort.

Convergence in these analyses is not a primary concern but the results can be highly sensitive to plasticity models and mesh size. The kinetic energy of the dropped object is converted to strain energy in the struck and the striking bodies. These analyses should be accurate in the defined impact velocity and the dissipation of strain energy through the developed plasticity. Material failure of the struck body in the form of rupture or crack is the primary concern as the impact can escalate to secondary hazards (gas dispersion, fire, or explosion). Different impact angles should be considered, often in a parametric style of definition, as this can influence plate rupture. In a numerical analysis when using solid elements the assumption of a head-on impact (theta = 90deg) and impact at an angle (theta ≠ 90deg), can be the difference of a surface-to-surface type of contact in the former case, while in the latter case can be an edge-to-surface type with highly localised strains that drive failure. The above contact definitions are not a concern when shell elements are used to model the stiffened plate and the dropped object.

The above paragraph concerns striking and struck bodies in air. The striking velocity of bodies that experience free fall in water is different than in air ($v = \sqrt{2gh}$) and needs to consider their mass and area, the height h in air prior to contact with water, the hydrodynamic added mass, water density and drag coefficient. Guidance is given in DNV-RP-C204 (2010) but the subject of sinking dynamics deserves a thorough treatment.

6.4.3   Explosions

Explosive loads on fire walls, louvered panels and bulkheads are usually considered uniformly distributed over the exposed structure on a component level analysis. For global response analysis the overpressure distribution is highly non-uniform with local maxima and minima throughout the structure. A global analysis can reveal multiple locales where overpressure pockets are developed and numerous cases could be studied as a result. In design level recommended practices can approach this type of loads deterministically, by means of a suitable shape function for the displacements in the elastic and elastic-plastic range. Several assumptions are made in the boundary conditions of explosively-loaded panels and the system's fundamental periods. Recommended practices are based on Bigg's theory with its suitability discussed by Heywood and Martland (1999). Symmetric triangular pressure-time profiles, characterised by rise time, maximum pressure and pulse duration, are often preferred for dynamic structural analyses. Several other idealisations can be applied (rectangular, linearly decaying, gradually increasing, but

practitioners should be aware that nonlinear structural response can be different depending on the blast load profiles. Applying uniformly distributed loads on design may be inadequate in terms of capturing a representative response as deformations tend to be underestimated. A more accurate, non-uniform, pressure application based on coupled CFD-structural models is presented in Paik et al. (2014) where the torsional behaviour at the topside connection is considered and compared to results without any connections. The main benefit from integrated analyses is the enhanced precision with which the nonlinear structural response, with time and location-dependent explosive loads, is performed.

Models of blast resistant members like blast walls, bulkheads, decks and partition walls need to include their supporting members (stools, shedder plates and brackets) onto the deck plating where they are supported. These structures are often modelled with shell elements while beam elements may be present as well. Due to the high strain rates experienced by the material, strain rate effects need to be considered as well as temperature dependency. NORSOK N-003 and DNV-RP-C208 suggest the use of temperature dependent stress-strain relationships as given in Eurocode 3, Part 1.2, Section 3.2 (EUROCODE, 2005).

6.4.4   Fire

As it is not feasible to design for a worst case scenario accounting for a prohibitively high number of cases, probabilistic approaches based on quantitative risk analysis (QRA) are often implemented. A formal QRA often provides the necessary input in the form of leak location, wind direction, leak rate (kg/sec) and ventilation that help in determining the number of cases for analysis. A specific leak rate can yield numerous scenarios for analysis as it returns different consequences and different leak rates form explosive atmospheres with different frequencies of ignition. A successful QRA will also consider escalation of events that might lead to explosion through a combination of probabilistic fire analysis, CFD simulations, and thermal coupling to the structural model.

The use of FE analysis is at most times the preferred option due to the coupled effects that thermal loads induce to the structure particularly when originated from accidental scenarios. Software packages often analyse the accidental scenarios where a combustible plume is formed based on a CFD code and ignited based on probabilities identified in QRA. The structural assessment is done based on the coupled solution between a hydrodynamic code and the structural model. The output of these analyses takes the form of predicting the collapse of the structure when subjected to fire. A new procedure that introduces and benchmarks a coupling tool between KFX and Ls-Dyna is presented by Paik et al. (2013). The main benefit from this integrated analysis is the enhanced precision with which the nonlinear structural response, with time and location-dependent heat loads, is performed. This advantage is in line with a later study from Paik et al. (2014) where the benefits of integrated analysis are highlighted.

Different methods to apply heat loads are available. Thermal loads defined in the form of heat flux (radiative and convective) from hydrocarbon fires such as gas jet fires and pool fires, based on specific mass release rates (kg/sec), can be found in NORSOK-S-001 (2008) and FABIG TN11(FABIG, 2009). Standard fire/time temperature curves ISO-834-1 (1999) and EN 1363-1(EUROCODE, 1999a)) are typical curves for cellulosic combustibles (wood, textile). A hydrocarbon curve ISO-834-2 (2009) and EN 1363-2(EUROCODE, 1999b)) can also be used to simulate hydrocarbon fires if required. Additional input for hydrocarbon fires can be found in UL1709(UL, 1994) and ASTM E1529(ASTM, 2014) where the temperature rises rapidly and levels out at 1093°C for the remaining of the simulation. These thermal loads are to be applied either via using directly the temperature as a boundary condition, via heat flux boundary conditions, or using radiation and convection with prescribed ambient temperature history.

Thermal analyses are solved with an implicit solver in time domain. Explicit solvers should be avoided as unwanted response will be registered and analysis time can be prohibitive for long duration events in the time frame needed.

The appropriate temperature-dependent properties of the materials involved in thermal analyses need to be considered. Steel and aluminium have different properties particularly after the heat load application, while most aluminium alloys are more vulnerable than steel when exposed to elevated temperatures (see Paragraph 6.5 for further details).

6.5   *Material Models*

ISO-19902 (2007) dictates that the expected non-linear effects, including material yielding, buckling of structural components and pile failures, should be adequately modelled and captured. Carbon steel is the most popular of the materials used in ship and offshore structures. Stainless steel is also found particularly in the topside structure together with aluminium alloys.

Material properties vary with the carbon content that affects ultimate strength, yield strength and tensile failure levels. Carbon steel exhibits a linear stress-strain relationship until the material's yield strength, unlike stainless steel that exhibits a rounded response in its stress-strain curve adding uncertainty

to the extraction of a yield strength value. Generally, when using stainless steel more than two points should be used in the stress-strain curve around yield strength. This difference makes the use of design rules necessary so that uncertainty is reduced and industry accepted values are referenced in the analysis.

Material properties will also vary in thermal analyses where thermal properties need to be considered as a function of temperature. Significant differences in the thermo-physical properties (conductivity, specific heat capacity and thermal strain) of carbon steel and aluminium are noted when exposed to high temperatures; changes in the mechanical properties (Young's modulus and yield stress) are also taking place and should be addressed. More specific properties for different steel qualities are found on SCI (2006), FABIG TN11(FABIG, 2009), FABIG TN6(FABIG, 2001), and Eurocode 3 (EN 1993-1:2(EUROCODE, 2005)). Properties for aluminium alloys can be found in Eurocode 9 (EN 1999-1:2(EUROCODE, 2007)). A succinct pool of information for materials used in the context of ALS design can be found in the Material Database presented in this work in Annex 1.

For conducting a thermal analysis, the thermal properties of the materials have to be included in the analysis. These include the thermal expansion parameters and the reduction of mechanical properties as a function of temperature. For common metallic materials, the typical values for these properties can be found from many engineering handbooks, material databases and design rules. For steel and other metallic materials the typical properties to be included as a function of the temperature are the following:
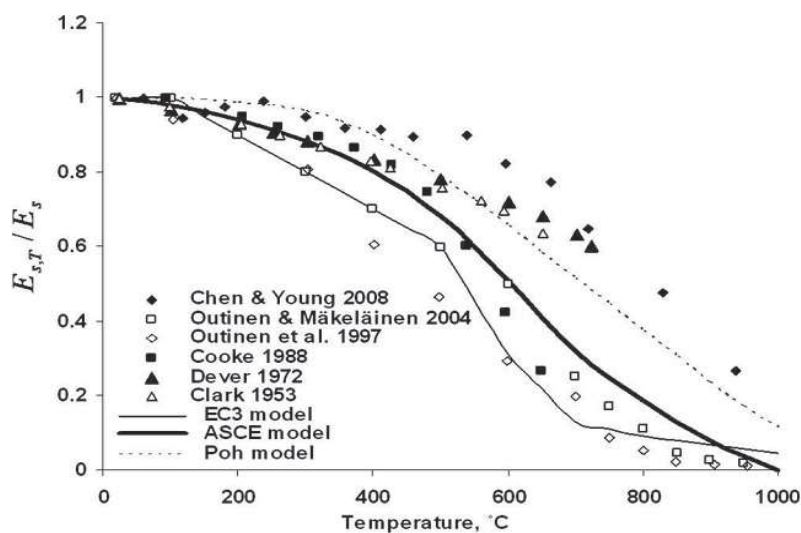
Reduction of Young's modulus

Figure 18. Elastic modulus of steel as predicted by different models and as measured in different tests (Kodur et al., 2010).
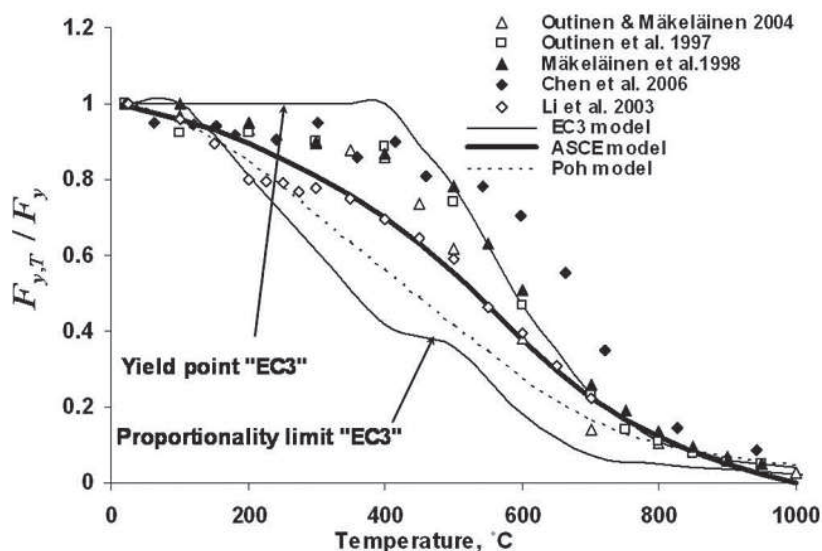
Reduction of Yield Strength

Figure 19. Yield strength of steel as predicted by different models and as measured in different (Kodur et al., 2010).
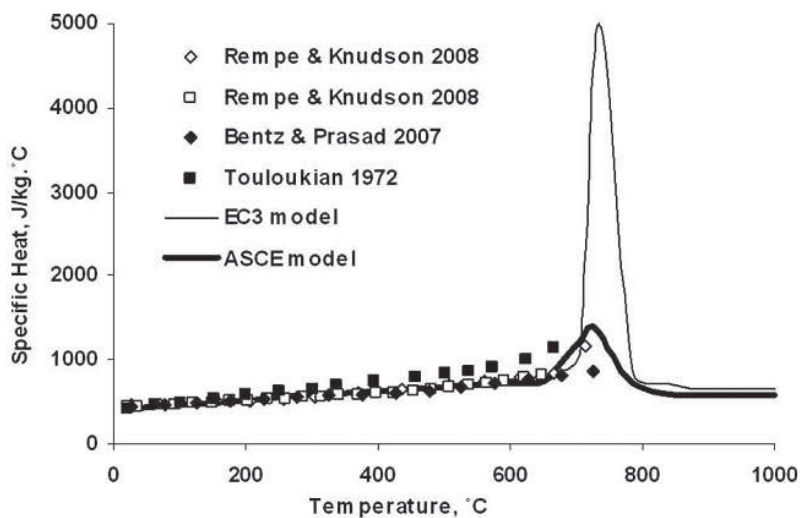
Specific Heat Capacity

Figure 20. Specific heat of steel as predicted by different models and as measured in different tests (Kodur et al., 2010).
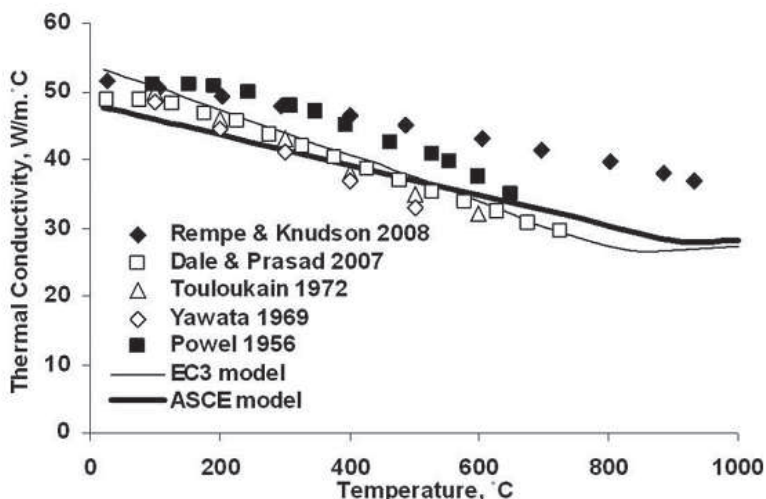
Thermal Conductivity

Figure 21. Thermal conductivity of steel as predicted by different models and as measured in different test programs (Kodur et al., 2010).
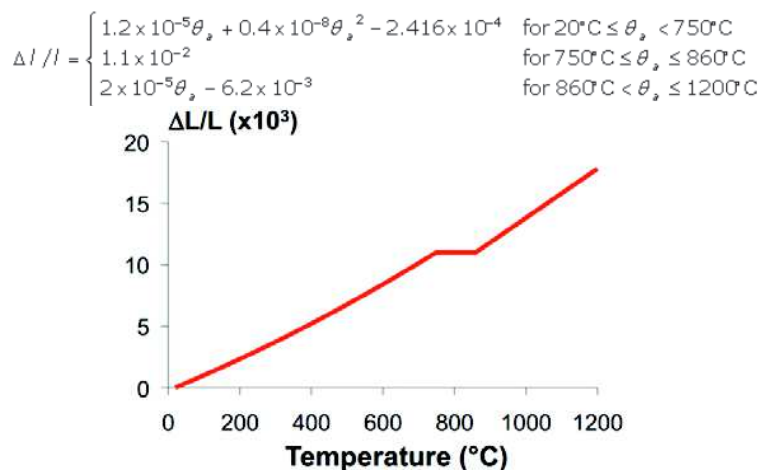
Thermal Elongation

$$\Delta l / l = \begin{cases} 1.2 \times 10^{-5}\theta_a + 0.4 \times 10^{-8}\theta_a^2 - 2.416 \times 10^{-4} & \text{for } 20^\circ C \leq \theta_a < 750^\circ C \\ 1.1 \times 10^{-2} & \text{for } 750^\circ C \leq \theta_a \leq 860^\circ C \\ 2 \times 10^{-5}\theta_a - 6.2 \times 10^{-3} & \text{for } 860^\circ C < \theta_a \leq 1200^\circ C \end{cases}$$

Figure 22. The coefficient of thermal elongation of steel.

Surface Emissivity
Temperature dependent, typically 0.25–0.8.
   From the modelling point of view, three material characteristics are of primary concern, namely the plasticity model, the stress-strain curve, and the defined failure criteria; these are discussed in the following paragraphs.

6.5.1   Plasticity Model

Plasticity is a property that has to be considered with the corresponding hardening model (kinematic, isotropic, or a combination of both). Steel is generally known to exhibit kinematic hardening. In analyses where the ultimate strength is sought, a piecewise linear isotropic plasticity model can be used while kinematic hardening is more appropriate to cyclic loading. Permanent displacements from impulsive loads are known to be unaffected by the hardening model; typically a combination of kinematic-isotropic is being used (β=0.5). A wide-ranging research on the effects of high strain rates on material properties is given in OTI 92 602.
   Plasticity with hardening exhibited beyond the yield strength can be described with a number of expressions. Five common models in FE analysis are:

a. Elastic-perfectly plastic
b. Elastic-linear hardening
c. Elastic-piecewise hardening
d. Elastic with exponential hardening
e. Ramburg-Osgood equation

   Most FE codes will assume that true values (not engineering) are being used in the material input data. It is the responsibility of the engineer to convert engineering to true values; guidance is provided in DNV-RP-C208 and in Appendix A. As a rule of thumb, true stresses will be higher than engineering (nominal) when plotted against strain.
   Typical yield strength values for carbon steel are in the range of 420MPa with ultimate strength in the range of 500-600MPa and failure strain of 19%. For stainless steel the corresponding values are 460MPa, 640MPa and 25%. Modulus of elasticity can be considered in the range of 200-210GPa with Poisson's ratio of 0.3. The variation on material strength as a result of thickness change should also be considered in the analysis; values can be found in EN 10025:3(EUROCODE, 2004). Additional information can be found in the Material Database presented in this work in Annex 1.

6.5.2   Stress-Strain Curve

Acceptance criteria often dictated by classification societies determine the material properties or provide some guidance as to which material models should be used in non-linear analyses. In ALS design it is highly recommended that the material properties are obtained from appropriate material certificates, or laboratory tests, which present the whole stress-strain curve that can be converted to true strains and stresses and used as a basis to define a failure criterion. In lack of specific figures the ultimate strain can be taken as 15%, or $100 \times$ strain at yield point ε_y (Eurocode 3). Failure based on plastic strain should always consider mesh sensitivity and strain rate effects.
   In impulsive loads where the material undergoes large degrees of membrane stretching, shell thinning needs to be considered as the plasticity level increases. This is usually combined when true stress-strain values are used; in the opposite situation where engineering values are used, it is more appropriate to neglect membrane stretching.

6.5.3   Failure Criteria

ALS design often involves simulating non-linear deformations and ductile fracture in large complex structures. In these analyses, shell elements are preferred over solid elements for computational reasons. A key question in such analyses is to establish a proper material curve for plastic region and a reliable fracture criterion that could cover the length scales from large structural shell elements to the material behaviour in the crack tip including localization in the form of necking. There are two main reasons, which complicate establishing a reliable fracture criterion (Kõrgesaar et al., 2014):

−   Shell elements with plane stress formulation are incapable of representing the 3D stress state during necking and localisation of deformation preceding fracture initiation.

− Large shell elements with an element length L_e ($L_e = \sqrt{A}$, where A is the element area) of several times the sheet thickness cannot capture the high strain gradients in the localization band, which reduces the accuracy even more.
− Localisation in the form of necking causes mesh size effects, whereby fracture strain increases, as the mesh is refined.

For a practical approach to material modelling several topics presented above cannot be considered precisely. DNV-GL has recently published and is still developing further the recommended practices for the determination of structural capacity of non-linear FE analysis methods (DNV-RP-C208). Among other issues the DNV-RP-C208 suggests material curves both for engineering stress-strain and true stress-strain space. The material curves are defined until the necking that is assumed to initiate once the ultimate strain value is reached, see Figure 23. Several studies have shown that after the material possesses significant resistance after the necking and the material curves and the failure strain should be scaled according to the element size, see for example Ehlers and Varsta (2009) in Figure 23 and Hogström et al. (2009) in Figure 24. Typical approach is to calibrate a constant failure strain (for example using an empirical expression by Scharrer et al. (2002) after which the fracture is assumed to occur and the element is removed from the simulation.
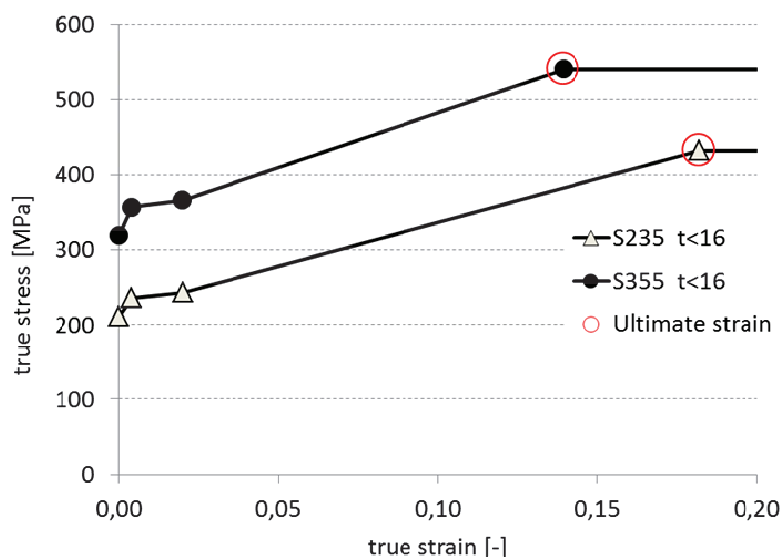

Figure 23. DNV-GL true stress-strain curve for S235 and S355 (DNV-RP-C208, 2013).
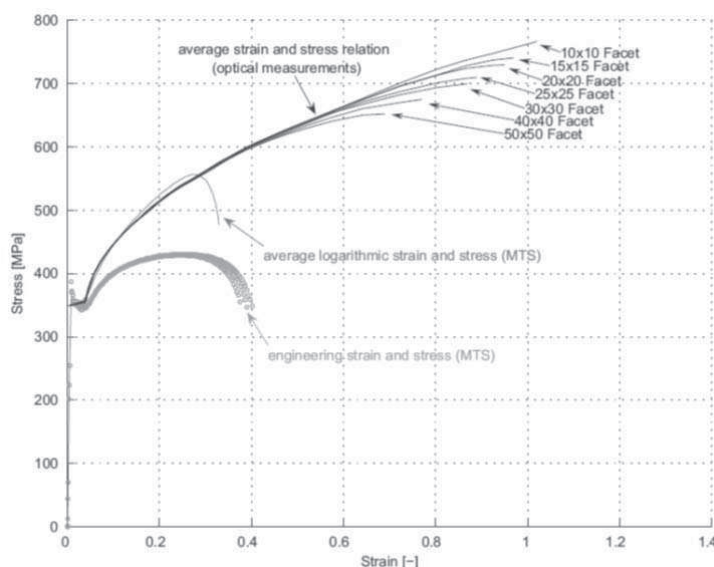

Figure 24. True stress-strain curve obtained by optical measurement (Ehlers and Varsta, 2009).
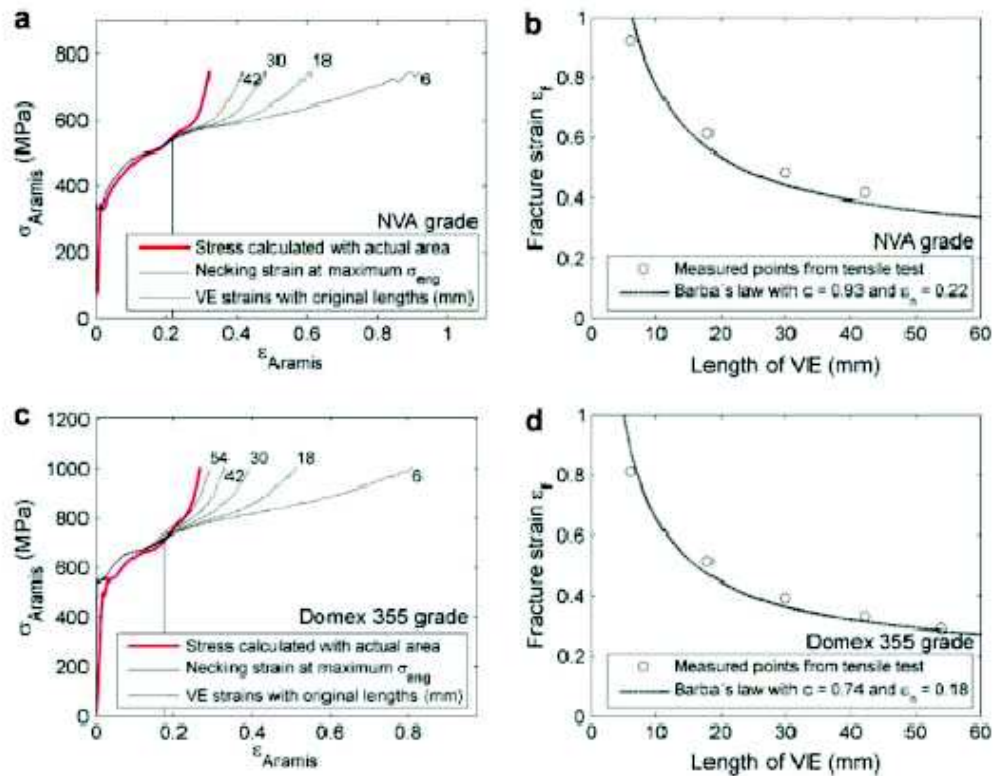
Figure 25. True stress-strain curve obtained by optical measurements for NVA and Domex 355 grade steel (Hogström et al., 2009). VE stands for Virtual Extensimeter and describes the length over which the stress-strain relation is evaluated. Smaller VE value corresponds to smaller elements.

While the constant ultimate plastic strain is a popular criterion, more advanced failure criteria and scaling models are being developed. Employing equivalent plastic strain measured with tensile tests as a fracture criterion and scaling it based on some law (See Barba's Law or some empirical relation) to account for different mesh sizes, is sufficient in cases where the stress state corresponds to uniaxial tension and does not change significantly during the analysis. However, in real structures such idealistic conditions rarely occur. Under multi-axial stressing, fracture depends on the pressure or the stress triaxiality as noticed by several authors and shown in many experimental studies, see Bao and Wierzbicki (2004), Barsoum and Faleskog (2007), Wierzbicki et al. (2005), Haltom et al. (2013), Hopperstad et al. (2003). Bai and Wierzbicki (2010) showed that depending on the stress state, the fracture strain could vary several times for the same element length. Therefore, a reliable fracture criterion used for large structures must incorporate the effect of the stress state in addition to element size. One of the criteria accounting for that, available in LS-Dyna is the so-called Rice–Tracey–Cockcroft–Latham criterion (RTCL) presented by Törnqvist (2003). This criterion includes the effect of the stress state, but the calibration of the criterion to account for the mesh size effects is solely based on the equivalent plastic strain determined with the uniaxial tension test. In the same criterion the fracture strain can be adjusted for different element lengths according to Barba's Law without the influence of the stress state.

   A novel theoretical framework for adjusting failure strain based on the stress triaxiality and element size has been given by Walters (2014). Kõrgesaar et al. (2014) studied the validity of Walter's adjustment of the fracture strain by exploring its ability to model fracture in large-scale shell structures by using Walter's framework to calibrate a stress triaxiality based fracture criterion formulated by Lou et al. (2012). Calibrated criterion was used in simulating large-scale panel indentation experiments carried out by Alsos and Amdahl (2009). The comparison of the measured force-displacement curves with simulated ones indicates that the approach of scaling fracture strain based on both, stress state and mesh size, accurately accounts for the reduction in ductility with increasing element size. Furthermore, comparison with the fracture criterion adjusted only based on the uniaxial tension test demonstrated that the new approach reduces the element size dependency. Kõrgesaar and Romanoff (2013) studied the effect of element softening while simulating the structural deformations using relatively large elements. However, these more advanced approaches by Walters (2014) and Kõrgesaar et al. (2014) are not yet implemented into commercial software packages.

6.6   Uncertainties of ALS Models

Addressing uncertainties associated with action effects should be given due consideration particularly for actions that require dynamic analyses where the structural response is nonlinear. Dynamic actions are highly sensitive to inertial and damping characteristics, particularly when the excitation source coincides with the natural frequencies of the structure. Specific actions like earthquake and ship collision relate to such sources where uncertainties are partly taken care of by safety factors. Numerous definitions on what uncertainty in modelling is exist in the literature, see Ayyub and McCuen (2011) and DNV (1992). In engineering analysis uncertainty in design can be addressed with probabilistic design analysis (PDA) tools; essentially a combination of probability theory and FE methods that can help estimating reliability indices. In very broad terms PDA provides an insight on the effect of loads and strength parameters, to the output values, e.g. plastic strain. By the time engineers set up a numerical model. Most of the random variables have been estimated, either from Class Rules, previous work, operator feedback, ongoing research, or open literature. The remaining uncertainties consist of those involved in model input parameters and those involved in modelling techniques. Modelling uncertainties can take the following form:

−   Structural discretization.
−   Material properties and models.
−   Description and application of loads.
−   Coupling between load and structural response (or coupling between action and action effects).

Efforts to reduce uncertainty in modelling and improve structural response are focused on using coupled models with integrated solutions like the ones presented by Paik et al. (2013), Paik et al. (2014) in fire and explosion analyses. Decoupled solutions can underestimate deformations, spatial distribution of heat and over-pressures or overestimate deformations occasionally. Decoupled solutions can also be inadequate in capturing the torsional behaviour at the topside connections and returning unreliable thermal loads.

Probabilistic models can address uncertainties based on reliability theory and introduce metrics like that of Robustness Index based on probability of failure under uncertain loads as presented in Czujko and Paik (2014). The advantage of probabilistic modelling of loads is that a frequency of load exceedance or, more precisely, frequency of exceedance of load parameters can be obtained. Uncertainties can take the form of exceedance curves represented by probability density function derived from curve fitting.

Lingering uncertainties remain in QRA undertaken prior to any analysis. These uncertainties will always remain in place as long as they are deemed to remain within allowable risk levels.

6.7   Probabilistic methods

Probabilistic structural analysis was defined by Ditlevsen and Madsen (1996) as being 'the art of formulating a mathematical model within which one can ask and get an answer to the question "*What is the probability that a structure behaves in a specified way, given that one or more of its material properties are of a random or incompletely known nature, and/or that the actions on the structure in some respects have random or incompletely known properties*?"

6.8   APPENDIX A

6.8.1   True stress-strain curve for Ls-Dyna

Typically, for non-linear finite element simulations true stress-strain curve is needed. For that engineering stress-strain curve obtained from the tensile test is transformed into true stress-strain curve. Here, a procedure is presented for a conventional tensile test. More advanced techniques based on optical measurements can be seen for example in Ehlers and Varsta (2009).

From the tensile test the engineering stress is obtained as a function of engineering strain. Engineering strain is transformed into true strain by

$$\varepsilon_{true} = \int_{L_0}^{L} \frac{dL}{L} = \ln(L) - \ln(L_0) = \ln(L_0 + \delta) - \ln(L_0) = \ln(\frac{L_0 + \delta}{L_0}) = \ln(1 + \varepsilon_{engin}) \qquad (A1)$$

where the following relations are used:

$$\varepsilon_{engin} = \frac{L - L_0}{L_0} = \frac{\delta}{L_0}$$

Equation (A1) is valid through the whole test process. Transforming engineering stress into true stress is more complicated than it was in the case of strains. As the cross-sectional area before the test, A_0, and after the test, A_T, are given, the true stress can be evaluated at the end of the test using relation:

$$\sigma_T = \sigma_Y \cdot \frac{A_0}{A_T} \qquad (A2)$$

where σ_y is yield stress ($\sigma_{0.2}$ for example). Equation (A2) can be used only at the last point of the test when broken specimen can be measured, as A_T values are not given for other test points. For the rest of the test region another approximation should be used. It is assumed that tensile pressure causes even decrease of cross-sectional area through the whole gauge length:

$$A_T(L + dL) = A_0 \cdot L$$

$$\Rightarrow \frac{A_0}{A_T} = \frac{L + dL}{L} = 1 + \frac{dL}{L} = 1 + \varepsilon_{engin}$$

Now the true stress can be approximated as follows:

$$\sigma_T = \sigma_{engin} \cdot (1 + \varepsilon_{engin}) \qquad (A3)$$

Equation (A3) is valid only until necking occurs and true stress values between the necking point and the last point are still undetermined. Undetermined values should be evaluated by using given true stress values. This is explained in Figure 26. Engineering stress-strain is obtained directly from the tensile test. As a next step, true stress value at the end of the test is calculated by using Equation (A2). In the graph it is marked with the circle at the far right of Figure 26. Approximation using eq. (A3) connects this extreme stress-strain value with the true stress-strain values before the necking, when the true stress-strain curve calculated from the engineering values starts to fall.
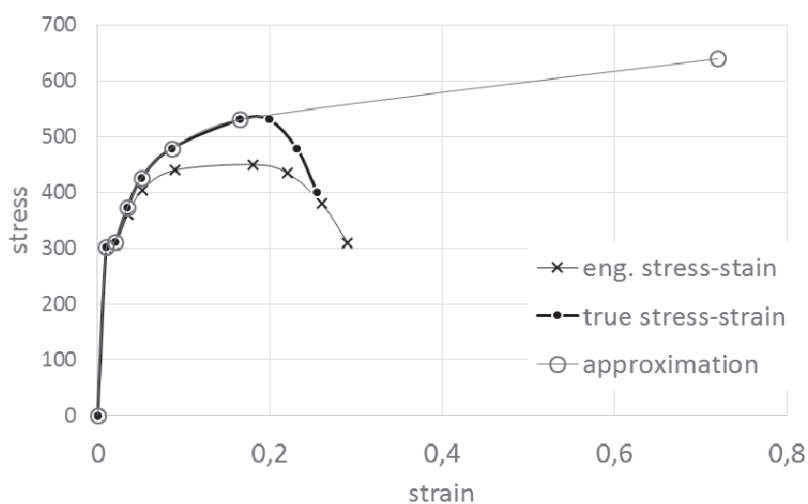

Figure 26. Stress-strain curves.

7. BENCHMARK STUDY. RESISTANCE OF TOPSIDE STRUCTURES SUBJECTED TO FIRE

7.1 Scope of work

The objective of the benchmark study is to predict the strength of topside structures subjected to fires and compare different techniques assessing the strength of these structures. The capabilities of modern software to simulate such complex loads are evaluated and Passive Fire Protection (PFP) design using numerical predictions are assessed.

The following committee members have contributed to the benchmark:

Table 7. Committee members contributed to the benchmark.

Participation	Affiliation	Analysis software	Reference on Figures
J. Czujko	Nowatec AS, Norway	USFOS, LS-DYNA	Czujko
L. Brubak	DNV GL, Norway	ABAQUS	Brubak
G.S. Kim	HHI, Korea	USFOS, LS-DYNA	Kim
K. Tabri	MEC, Estonia	LS-DYNA	Tabri
Y. Yamada	NMRI, Japan	LS-DYNA, Marc	Yamada
W.Y. Tang	Shanghai Jiao Tong University, China	ABAQUS	Tang
J. Czaban	Department of National Defence, Canada	LS-DYNA	Czaban

7.2   Strategy of benchmark study

The capability and accuracy of available techniques for the prediction of structural response of topside structures under fire loads are evaluated in terms of:

1. Temperature development on topside structures.
2. Deflections at 16 locations of the topside structure surface.

   The strategy of benchmark study is presented in Table 8.

Table 8. Strategy of the benchmark study.

Title	Descriptions
Primary study	1. Static analysis for all in-place loads
	3. Push-down analysis, uniform pressure until collapse
	4. Fire analysis
	4.1 Application of fire load defined by standard hydrocarbon fire curve
	4.2 Application of fire load by predefined constant global heat flux
	5. Design of PFP
Parametric study	1. Effects of boundary conditions
	2. Methods of controlling numerical instability for beam element model
	2.1 Element remove method
	2.2 Temperature remove method
	3. Effects of modelling assumption, shell vs beam element models
	4. Effects of local heat flux

7.3   Input

7.3.1   Geometry of target structure

Nowatec AS prepared a 3D CAD geometry model of topside deck structures to facilitate this benchmark study. The target structure was a part of an offshore topside deck structure, Figure 27. The thickness of deck plate was 8mm. In the first step the deck plate was not modeled explicitly but the weight of plate was included in the total weight of the deck. Geometrical details of the structure are given in Annex 2. All in-place loads acting on the deck structure were given to all participants in form of Excel and STAAD Pro input files.
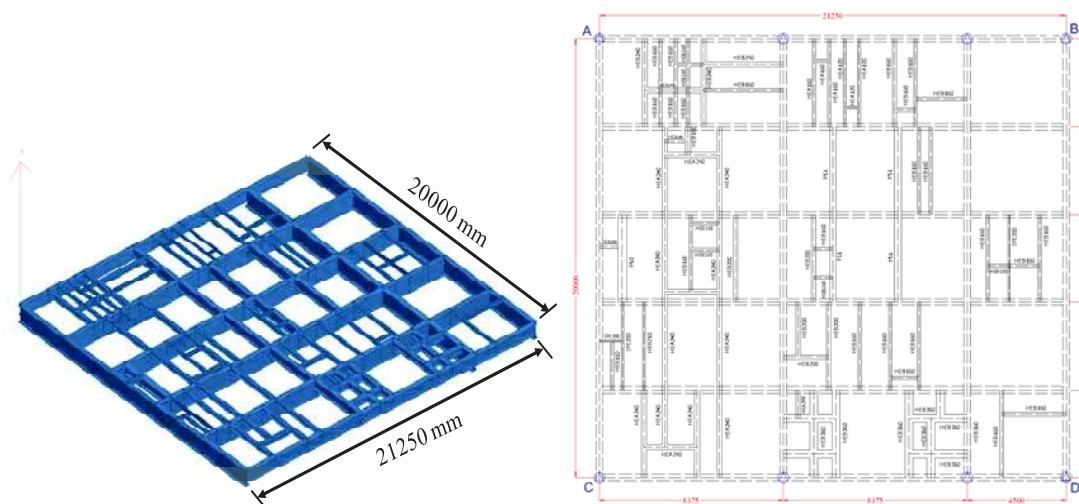

Figure 27. The deck structure. Geometry used in the benchmark study.

7.3.2 Material data

7.3.2.1 Strength properties of steel material

Material model for strength assessments was assumed to be bilinear elastic – plastic, described by the material constants.

European grade structural steel S355 was considered for this benchmark study. The yield stress and elastic modulus of steel at room temperature were 355 MPa and 210 GPa, respectively, and Poisson ratio was 0.3.

To perform the thermal and structural response analysis of structures under fire loads, material properties should be defined as temperature dependent. Figure 28 presents the reduction factors for yield stress and elastic modulus at elevated temperature defined according to Eurocode (EUROCODE, 2005).
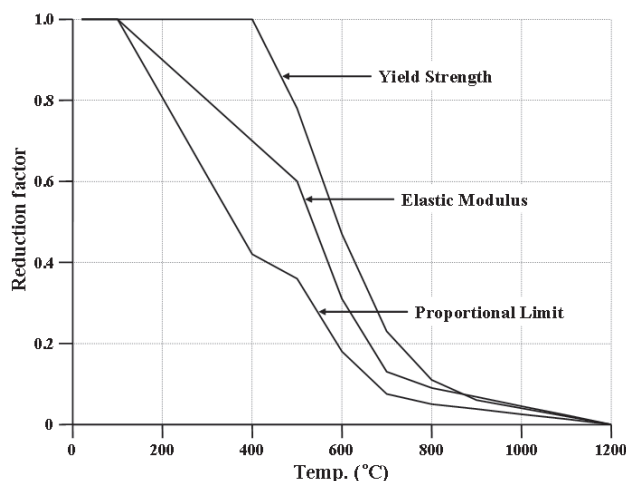
Figure 28. Reduction factors at elevated temperatures.

7.3.2.2 Thermal properties of steel material

The carbon steel material curves for elevated temperatures specified in Eurocode were used in the thermal and structural analysis. Figure 29 shows the curves for specific heat capacity and thermal conductivity, respectively (EUROCODE, 2005). The surface emissivity of steel was set to 0.8 for all structural members exposed to fire.
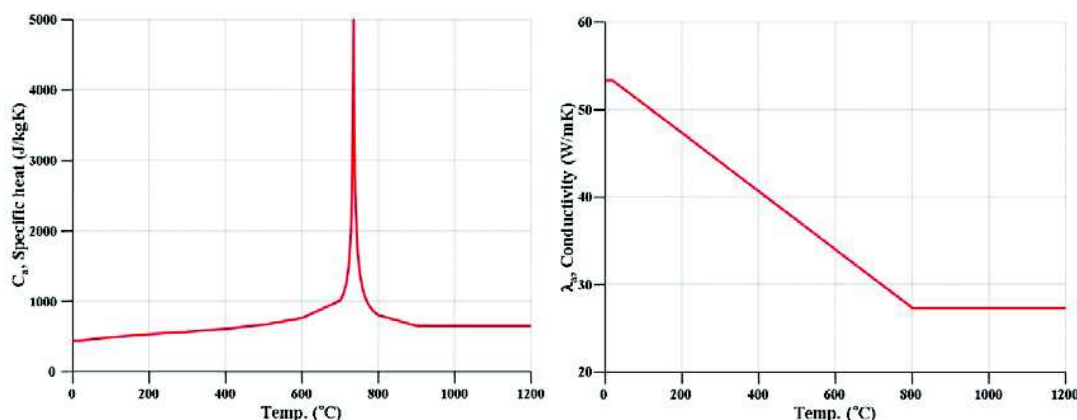

Figure 29. Thermal properties of steel: specific heat capacity (left) and conductivity (right).

7.3.2.3 PFP thermal characteristics

The fire protection material applied to a structural member was to remain unaffected and retain its fire performance if subjected to fire loads. The chosen spray-on fire protection material was Chartek VII. The thermal characteristics of Chartek VII shown in Figure 30 was used in this study (IPC, 2000).
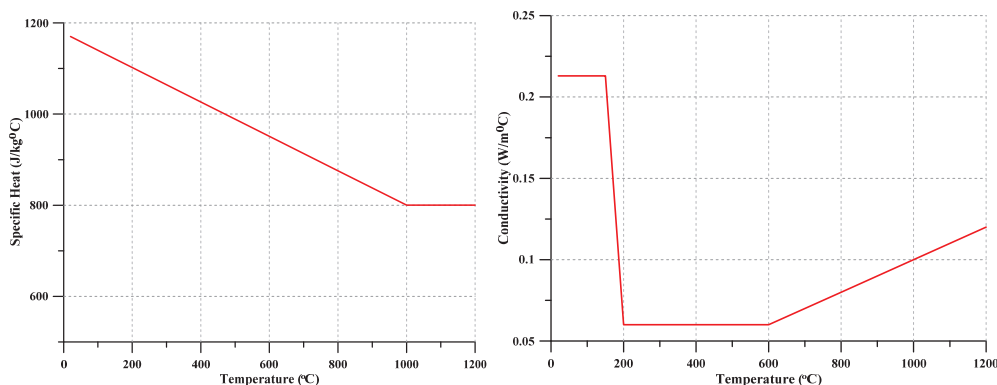
Figure 30. Thermal characteristics of Chartek VII: specific heat (left) and conductivity (right).

7.3.3   Boundary conditions

Figure 30 shows the structure was fixed at the supporting points (A, B, C, D, E, F, G, H) in the case of beam element model. Figure 31 and Table 9 illustrate the boundary conditions of finite element model. For finite element model, supporting points along the cross-section of the beam were all fixed. In this study simulations considering two types of boundary conditions, such as fixed in-plane (B.C_1) or free (B.C_2), were performed.
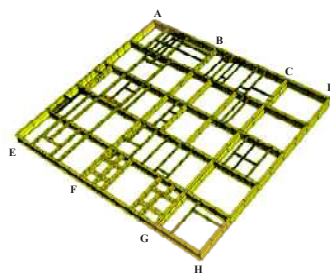

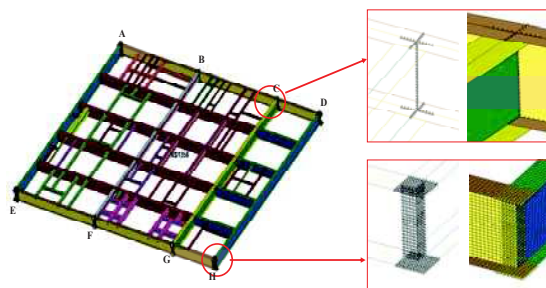Figure 31. Boundary condition of beam element model.

Figure 32 Boundary condition of shell element model.

Table 9. Summary of boundary conditions.

Boundary condition Descriptions (fixed =1)
B.C_1 The structure is fixed supported at the supporting points (A,B,C,D,E,F,G,H)
B.C_2   A (U_X, U_Y, U_Z, R_X, R_Y, R_Z =1) B, C (U_Z, R_Y, R_Z =1) D (U_Z, R_X, R_Y, R_Z =1) E (U_X, U_Y, U_Z, R_X, R_Y, R_Z =1) F, G, H (U_Y, U_Z, R_Y, R_Z =1)

7.3.4   Loads

7.3.4.1   Mechanical action

Three basic types of mechanical actions were applied in the model. However, in order to calculate the collapse strength of the deck model, push-down analysis was performed by using a push-down load case as shown in Table 10.

Table 10. Mechanical load case summary.

Load case Loading condition Based on Remarks
LC1 Gravity =1.1*9.81 (m/s2) Gravity loads should be applied individually. Mass of deck plate (27472kg) is considered in the gravity load
LC2 Live load =5.63 (kN/m2) live loads should be converted to shell and beam loads respectively
LC3 Piping and equipment Given in STAAD/USFOS piping and equipment loads should be applied as nodal loads (for application of LC3 see Annex 2)
LC4[*] Uniform pressure = Live load*scale factor Live load (LC2) is scaled up

*LC4 is used for push-down analysis

7.3.4.2 Fire action

The target deck structure was subjected to the following fire actions;
1. Standard hydrocarbon fire curve
2. Predefined constant global heat flux

In case of hydrocarbon fire, Eurocode provides the standard hydrocarbon fire equation as follows (EUROCODE, 2005);

$$\theta_g = 20 + 1080(1 - 0.325e^{-0.167t} - 0.675e^{-2.5t}) \tag{1}$$

where θ_g = gas temperature near the steel member in °C; and t = time in minutes.

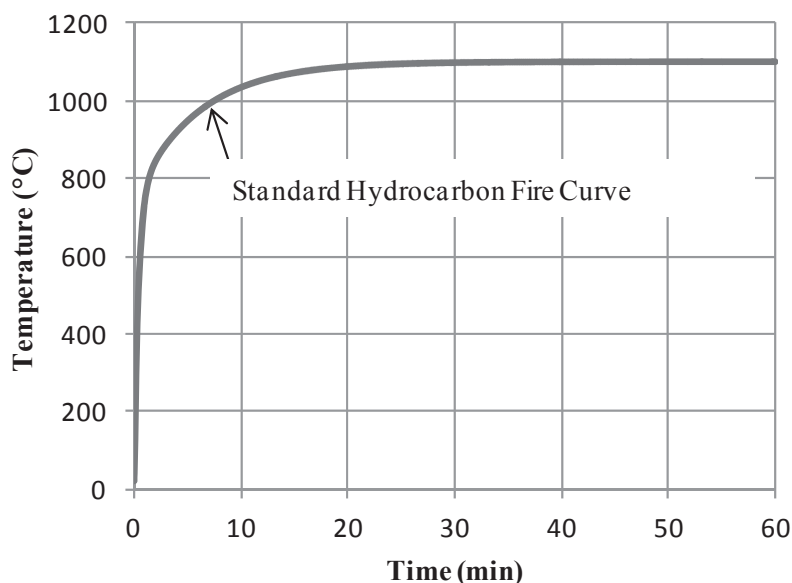Figure 33 shows the standard hydrocarbon fire curve.

Figure 33. Standard hydrocarbon fire curve.

In the case of constant global heat flux loading, the global heat flux on the target deck structure was Q_G=100 KW/m² and the local heat flux was Q_L=350 KW/m². Figure 35 shows the application area of global heat flux and Figure 34 presents the four locations of local heat flux on the target deck structure.
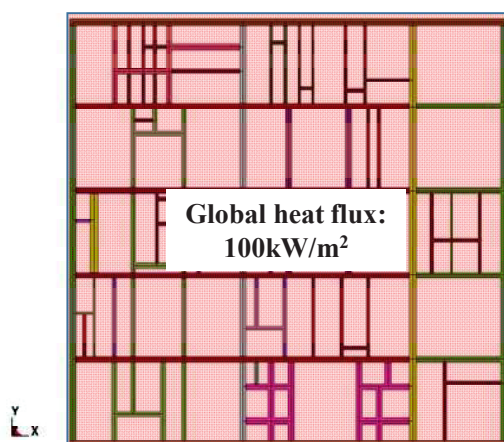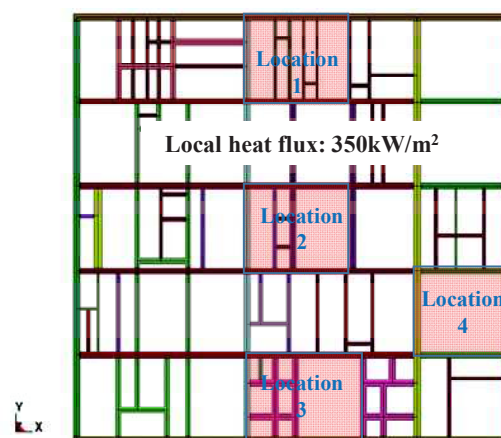

Figure 34. Global heat flux.

Figure 35. Local heat flux.

7.3.4.3 Monitoring of results

Figure 36 presents the location of monitoring points for reporting of deflections/damage during fire and residual deflections after fire over the target structure surface. The monitoring point considered was at the centre of the web of I-beam. From practical point of view, flange is more important, but it was decided to

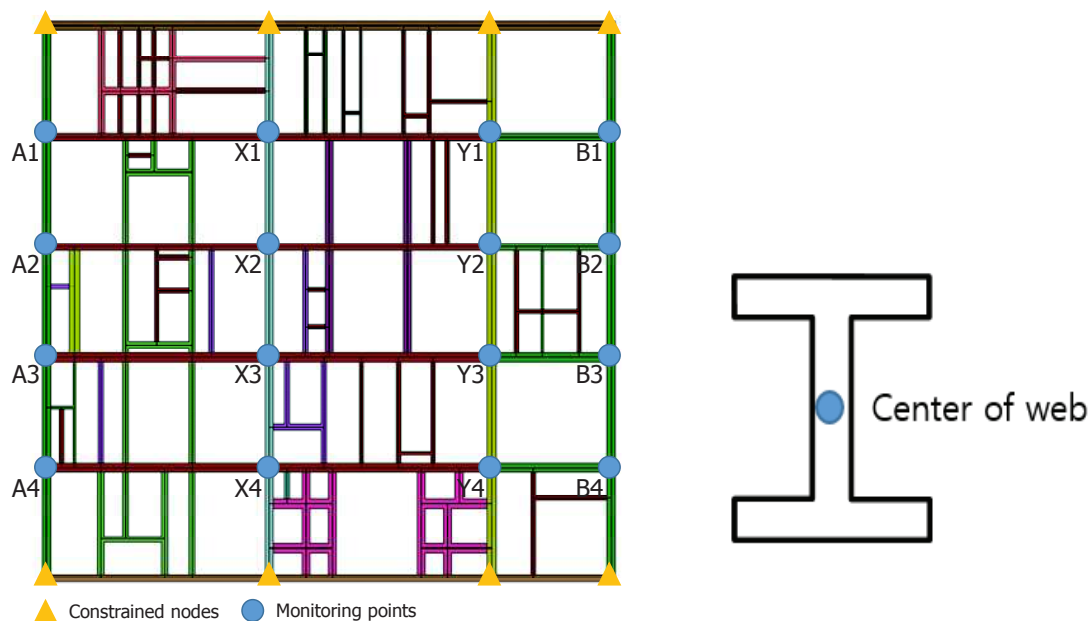report the results on the centre of the web for simplicity reasons and to compare with the beam element model.

Figure 36. Location of monitoring points on the target structure, deformation and temperature.

7.4   Results

7.4.1   Static analysis

Static analysis was performed by considering all in-place loads (self-weight, live load, piping & equipment load). Table 11 summarizes the modelling approach and reaction forces for in-place loads calculated by the participants of the benchmark study. Figure 37 shows the deflection at monitoring points due to in-place loads reported by all participants. Table 11 summarizes the mean and standard deviation at monitoring points.

Table 11. Modeling approach for benchmark study.

| Participant	Software	Element type	Boundary condition	Load		
				LC1	LC2	LC3
Brubak	ABAQUS	Shell	Fixed	7.31E+05	2.37E+06	3.38E+05
Czujko	USFOS	Beam	Fixed	1.02E+06	2.46E+06	3.38E+05
	LS-DYNA	Shell	Fixed	1.03E+06	2.46E+06	3.38E+05
Kim	USFOS	Beam	Fixed	1.04E+06	2.39E+06	3.38E+05
	LS-DYNA	Shell	Fixed	1.00E+06	2.37E+06	3.38E+05
Tabri	LS-DYNA	Shell	Fixed	7.34E+05	2.39E+06	3.38E+05
Tang	ABAQUS	Shell	Fixed	7.27E+05	2.42E+06	3.38E+05
Yamada	LS-DYNA	Shell	Fixed	7.32E+05	2.48E+06	3.38E+05
	Marc	Shell	Fixed	7.32E+05	2.49E+06	3.38E+05
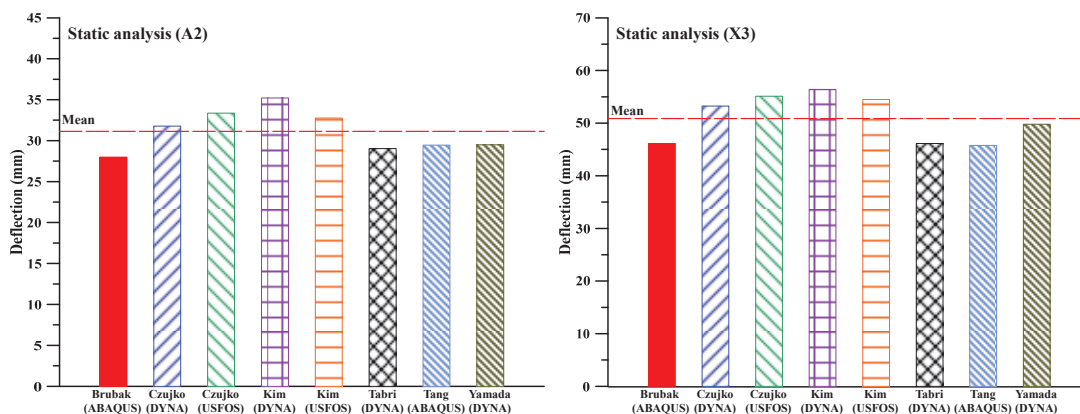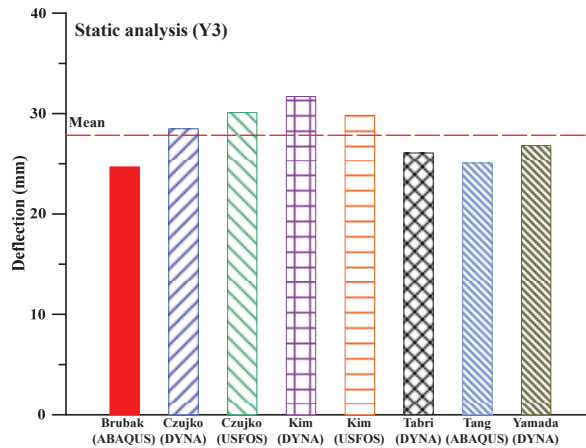
Figure 37. Deflection at monitoring points subjected to in-place loads.

Table 12. Modeling approach for benchmark study.

Monitoring point	Mean	Standard deviation
A2	31.14	2.36
X3	50.87	4.18
Y3	27.84	2.40

7.4.2   Push-down analysis

7.4.2.1   Shell element models

In order to calculate the collapse strength of deck model, push-down analysis was performed. All in-place loads (LC1~LC3) acting on the model did not induce the global collapse. Therefore, addition load (uniform pressure) was applied in the model and the collapse strength was reported by each participant. Figure 38 shows the comparison of results for push-down analysis among all participants using shell element model. The results indicate that the collapse strength of the deck model documented by each participant had good agreement with each other in the linear state, but had big differences in the plastic state. For any specific vertical force (e.g. 11600kN) the displacement reported by each participant was compared and the results differed about 20% with the mean value, which is significant.
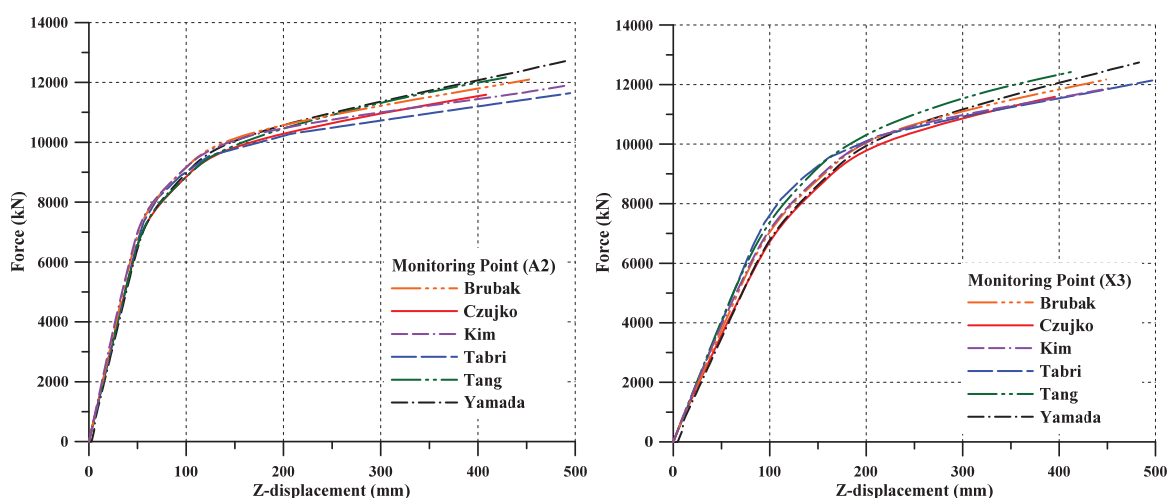

Figure 38. Vertical force versus deflection at monitoring point A2 and X3, shell element model.

7.4.2.2   Shell vs. beam element models

Figure 39 shows the comparison of results for push-down analysis between beam element model and shell element model. The results shown in Figure 39 indicate that the beam and shell element models have good agreement in elastic state but beam element model collapsed early.
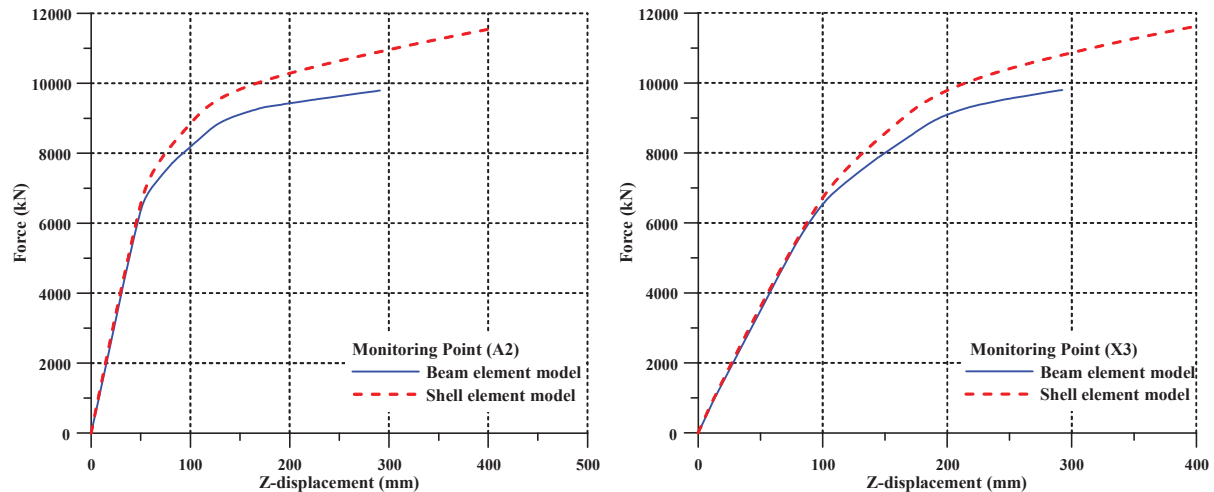
Figure 39. Comparison of results for push-down analysis between shell and beam element models at monitoring point A2 and X3.

7.4.3   Fire analysis

The objective of the benchmark study was to assess the capability and accuracy of available techniques for the prediction of structural response of topside structures subjected to standard hydrocarbon fire curve and constant heat flux loads.

7.4.3.1   Heat transfer analysis

Before to perform the structural response analysis subjected to fire, the results of heat transfer analysis should be compared. Figure 40 shows the results of heat transfer analysis for 30 minutes fires.

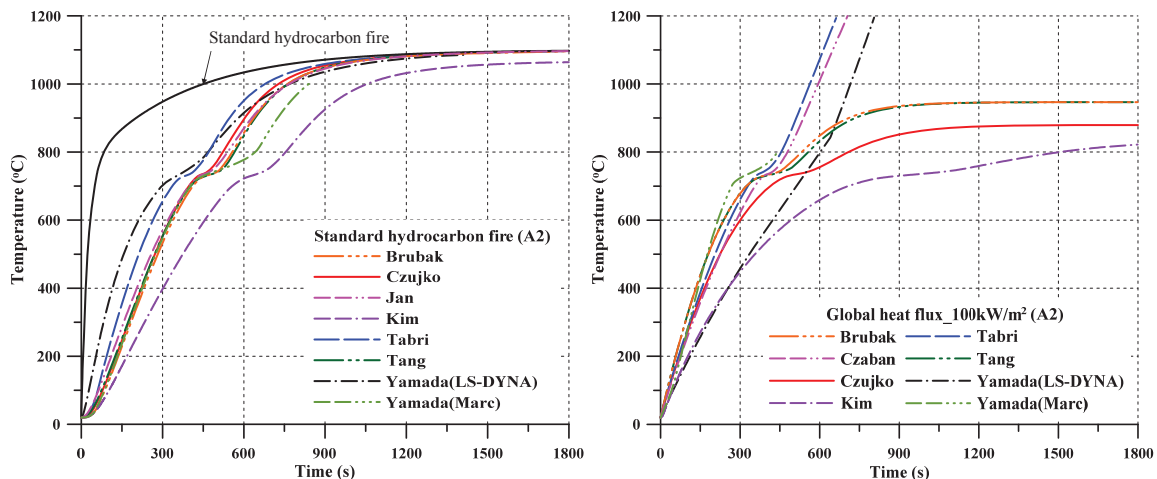Figure 40. Comparison of results of heat transfer analysis for 30 minutes fires at monitoring point A2.

7.4.3.2   Standard hydrocarbon fire curve

Shell element models
The loading condition was the application of standard hydrocarbon fire curve on the whole structure. Figure 41 shows the comparison of temperature and Figure 42 shows the comparison of deflection at monitoring point A2 and X3.
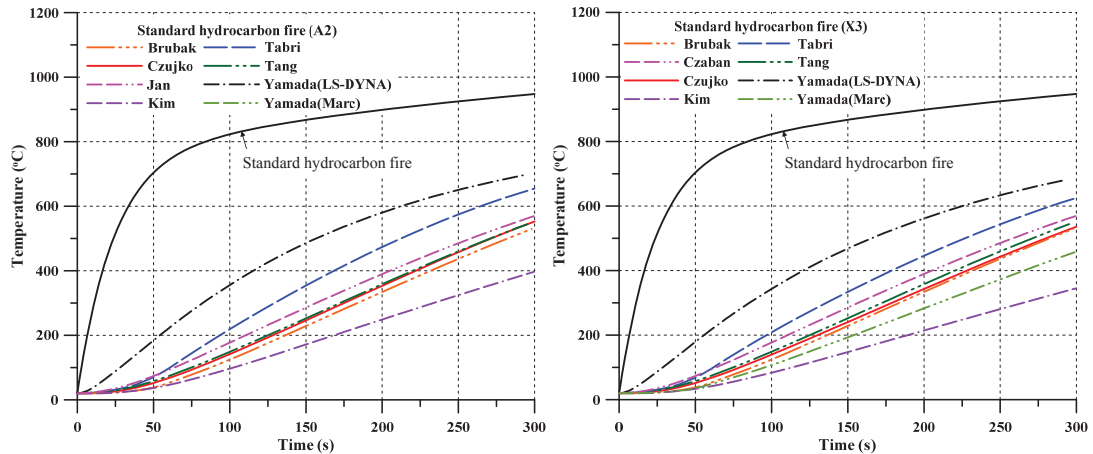
Figure 41. Comparison of temperature for standard hydrocarbon fire at monitoring point A2 and X3.

Figure 42. Comparison of deflection for standard hydrocarbon fire at monitoring point A2 and X3.

Shell vs beam element models

Figure 43 and Figure 44 show the comparison of results for standard hydrocarbon fire between beam element model and shell element model. The results of temperature and deflection have good agreement.

Figure 43. Comparison of temperature for standard hydrocarbon fire at monitoring point A2.

Figure 44. Comparison of deflection for standard hydrocarbon fire at monitoring point A2.

7.4.3.3   Constant global heat flux

Shell element models

The loading condition was the application of constant global heat flux ($100kW/m^2$) on the whole structure. Figure 45 shows the comparison of temperature and Figure 46 shows the comparison of deflection at monitoring point A2 and X3.
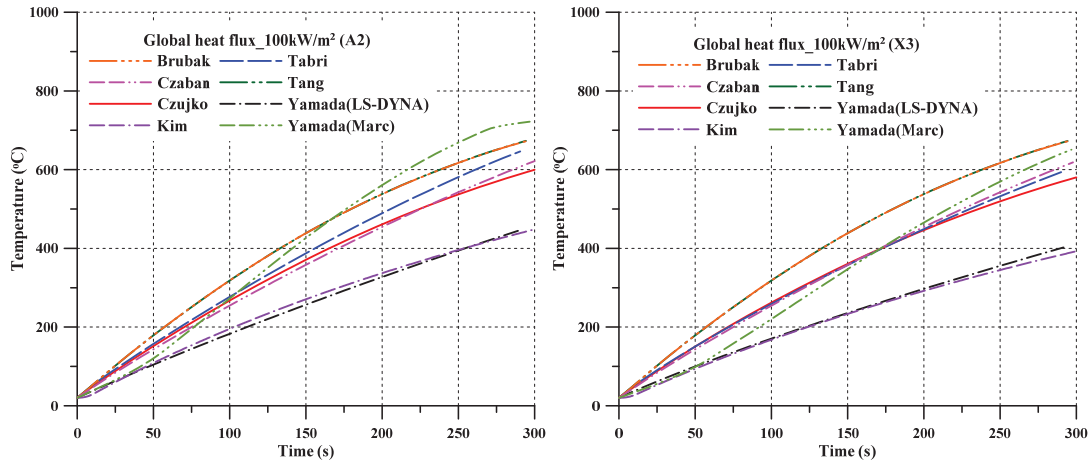
Figure 45. Comparison of temperature for constant global heat flux at monitoring point A2 and X3.
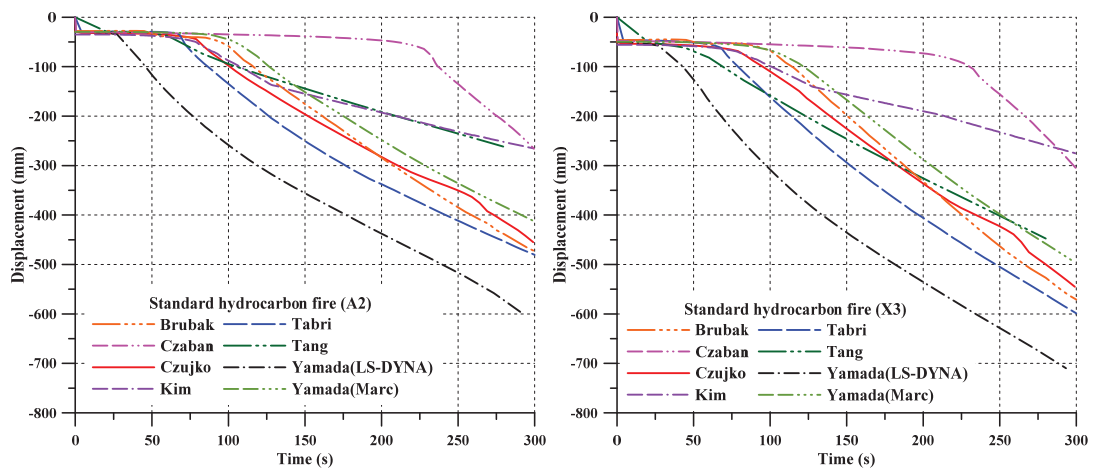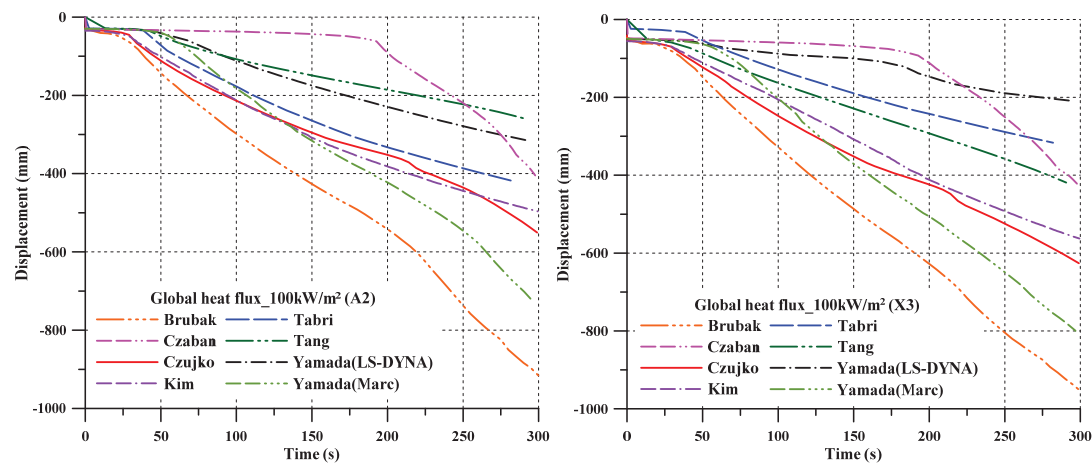

Figure 46. Comparison of deflection for constant global heat flux at monitoring point A2 and X3.

Shell vs beam element model

Figure 47 and Figure 48 show the comparison of results for constant global heat flux between beam element model and shell element model. The results of temperature and deflection have good agreement.
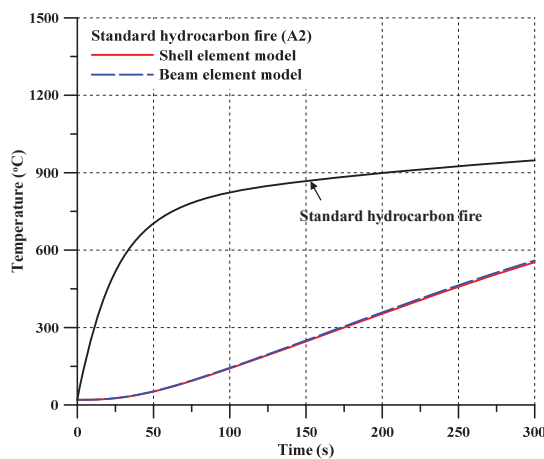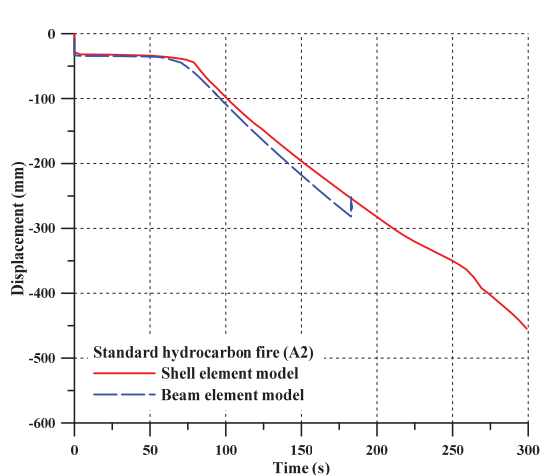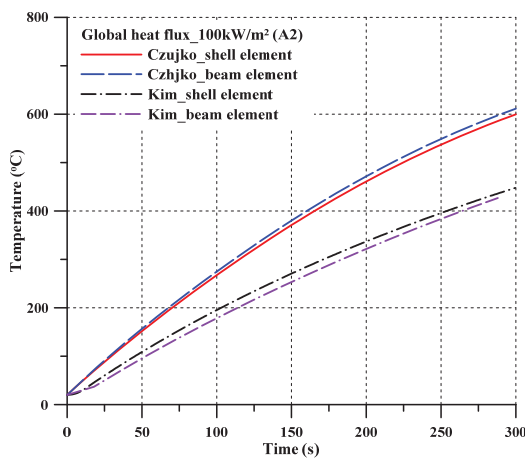

Figure 47. Comparison of temperature for constant global heat flux at monitoring point A2.
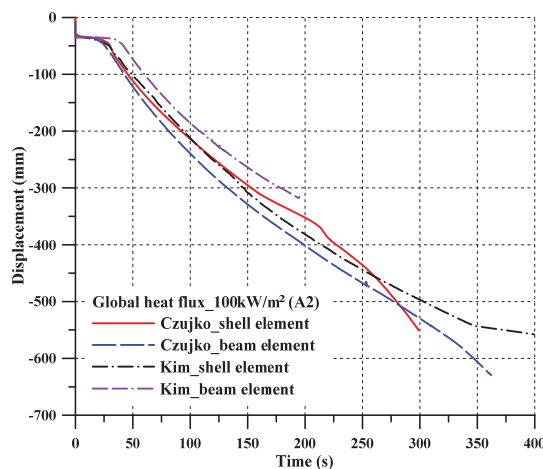
Figure 48. Comparison of deflection for constant global heat flux at monitoring point A2.

7.4.4   *Design of PFP*

In order to consider the effect of PFP, only the thermal properties were considered in the heat transfer analysis. In the case of USFOS program (USFOS, 2013), the input of PFP material is defined in terms of resultant heat conduction. Therefore, USFOS considers only the thickness and conductivity of PFP in

order to calculate the steel temperature of the PFP insulated structure. As specific heat capacity is not defined, it is assumed that there is no heat absorbed by PFP material. On the other hand, there is no possibility to calculate directly the effect of PFP in LS-DYNA program (LS-DYNA, 2014). One of the possible solutions is to make a model of PFP by solid elements and assign PFP properties on those solid elements. However, this procedure is very time consuming and cumbersome in the case of big and complicated FE models. Another possible way is to calculate the steel temperature through simple equation developed by Eurocode (EUROCODE, 2005) and apply that steel temperature in the FE model manually. The second procedure was adopted in the current benchmark study.

Figure 48 shows the comparison of temperatures for standard hydrocarbon fire with PFP at monitoring point A2 and X3. The deflection of standard hydrocarbon fire with PFP is presented in Figure 49.
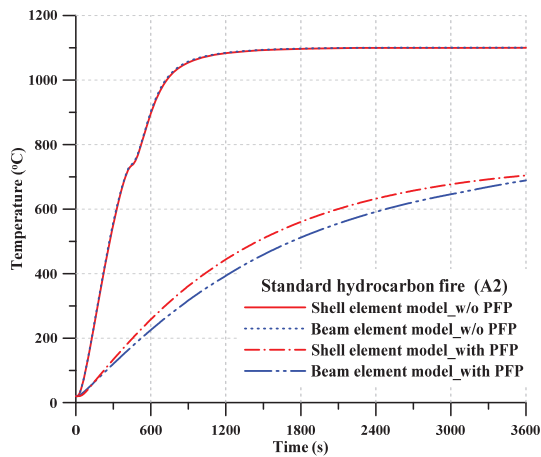

Figure 49. Comparison of temperature for standard hydrocarbon fire with PFP at monitoring point A2.
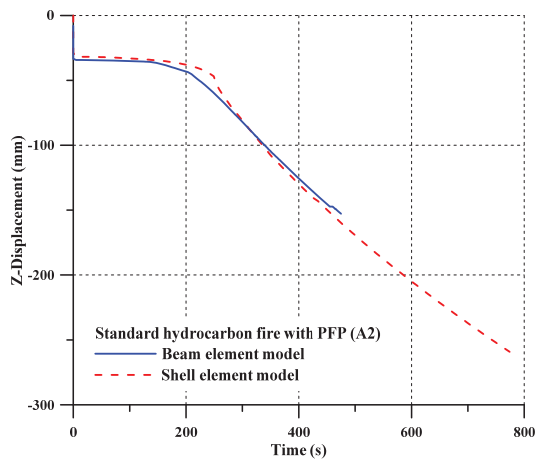
Figure 50. Comparison of deflection for standard hydrocarbon fire with PFP at monitoring point A2.

7.4.5 Effects of boundary conditions

The boundary conditions are very important for the proper calculation of the structural responses. In the case of beam element model, the center of cross section (node located at the center of the web) was used as the boundary node, and for shell element model the nodes located along the cross section of the beam were considered as boundary nodes (see Figure 51). Figure 51 presents the difference in deflection due to the boundary conditions. Standard hydrocarbon fire curve has been applied for this comparison study. The deflection is reported for monitoring point A2 and X3.
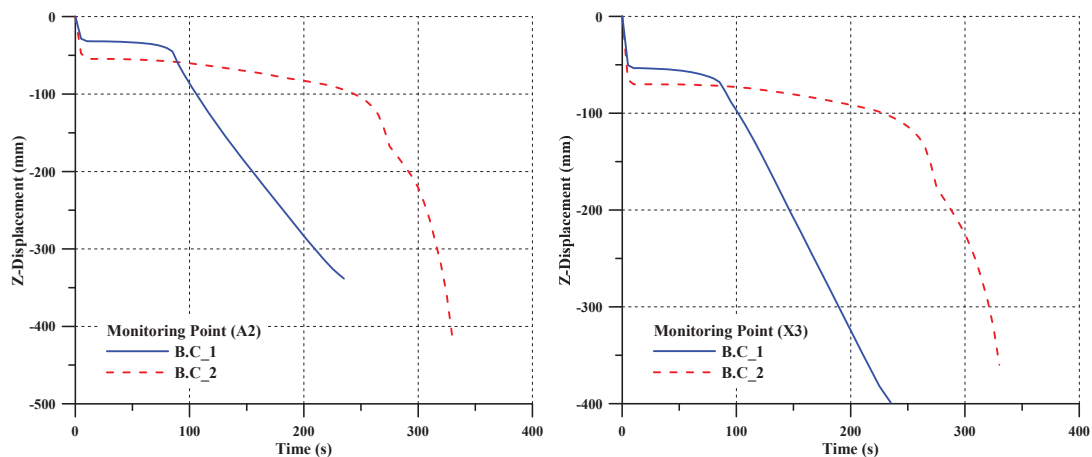
Figure 51. Comparison of deflection for standard hydrocarbon fire between B.C_1 and B.C_2.

7.4.6 Methods of controlling numerical instability for beam element model

In the case of beam element model, the numerical simulation sometimes stopped due to secondary member collapse. Loads at a secondary member resulted in large displacements and distorted elements, which caused the numerical problems. To solve this problem, two possible methods were adopted in the

benchmark study and compared with each other in terms of effectiveness and accuracy. The first one was named the element remove method and the second one the temperature remove method.

7.4.6.1 Element remove method

In this method the secondary members were removed when fire response analysis was performed. In the case of element remove method, the X and Y directional displacement was different than the original value at primary members as shown in Figure 52. Also, the global reaction force under in-place loads was different due to removed elements.

USFOS (Original)

USFOS (Elements remove)
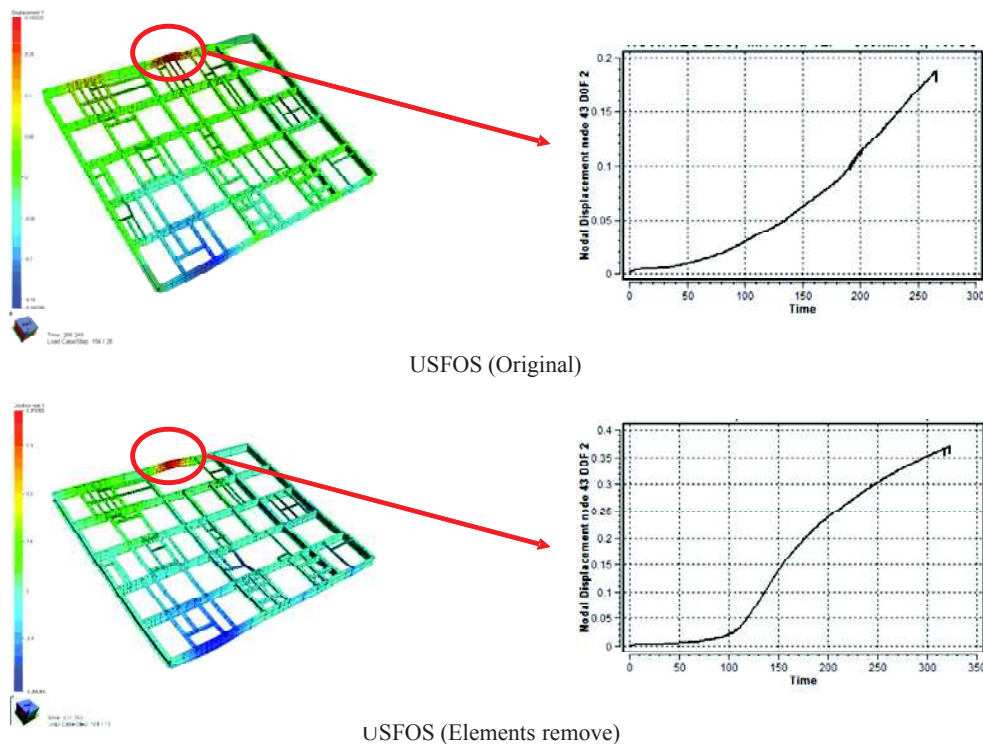
Figure 52. Comparison of deflection between original and elements removed model.

7.4.6.2 Temperature remove method

In the case of temperature remove method, the secondary members were not exposed to the fire loads. The heat transfer analysis was performed considering only the primary members exposed to fire and then structural response analysis was conducted using the results of heat transfer analysis. Figure 51 shows the comparison of deflection between element remove method and temperature remove method. In this benchmark study the temperature remove method was used in all fire analyses (both beam and shell element model) in order to keep consistency between beam and shell element model.
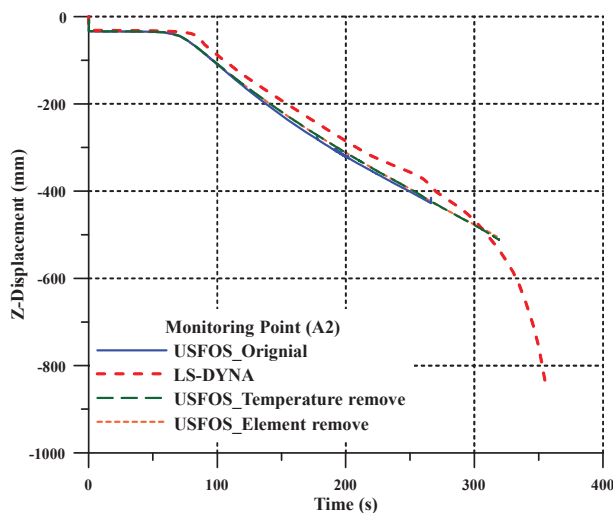
Figure 53. Comparison of deflection between element and temperature remove method.

7.4.7 Effects of local heat flux

7.4.7.1 Local heat flux (location 1)

This load case considers both global and local heat flux on the target deck structure. The local heat flux ($350kW/m^2$) was applied only in location 1 and global heat flux ($100kW/m^2$) was applied in the rest of deck structure. This case was analyzed to check the effect of local heat flux on the target deck structure. Figure 52 and Figure 54 show the results of local heat flux (location 1) analysis at monitoring point A2 and X1.

Figure 54. Comparison of temperature for local heat flux in location 1 at monitoring point A2 and X1.

Figure 55. Comparison of deflection for local heat flux location 1 at monitoring point A2 and X1.

7.4.7.2 Local heat flux (location 2)

This load case considers both global and local heat flux on the target deck structure. The local heat flux ($350kW/m^2$) was applied only in location 2 and global heat flux ($100kW/m^2$) was applied in the rest of deck structure. Figure 56 and Figure 57 show the results of local heat flux (location 2) at monitoring point A2 and X2.
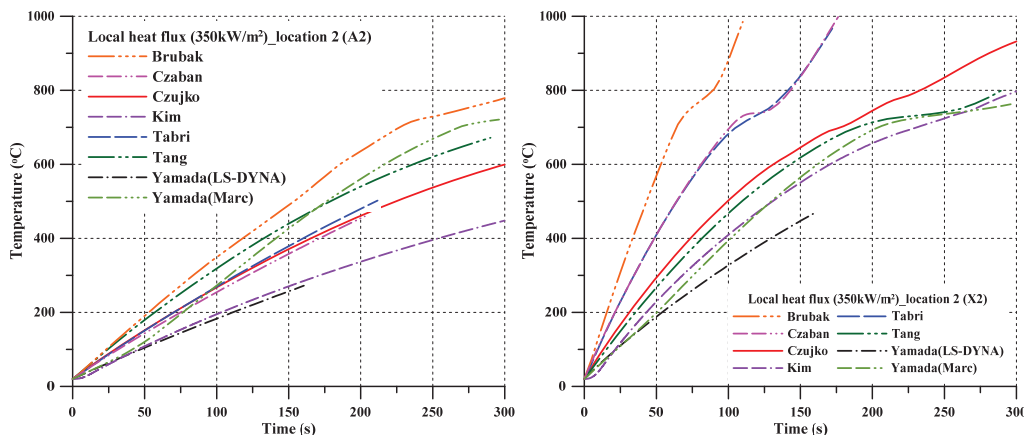
Figure 56. Comparison of temperature for local heat flux in location 2 at monitoring point A2 and X2.
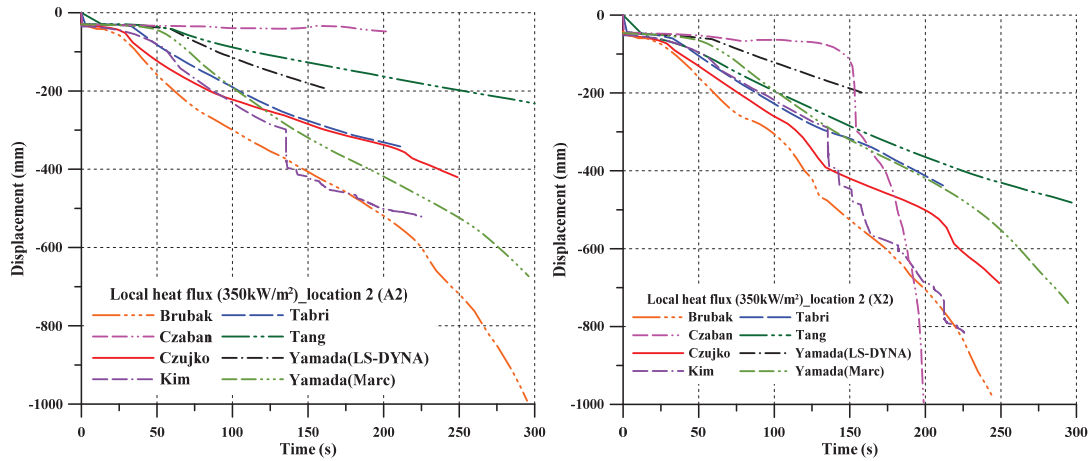
Figure 57. Comparison of deflection for local heat flux in location 2 at monitoring point A2 and X2.

7.4.7.3   *Local heat flux (location 3)*

This load case considers both global and local heat flux on the target deck structure. The local heat flux (350kW/m²) was applied only in location 3 and global heat flux (100kW/m²) was applied on the rest of deck structure. Figure 58 and Figure 59 show the results of local heat flux (location 3) at monitoring point A2 and X4.
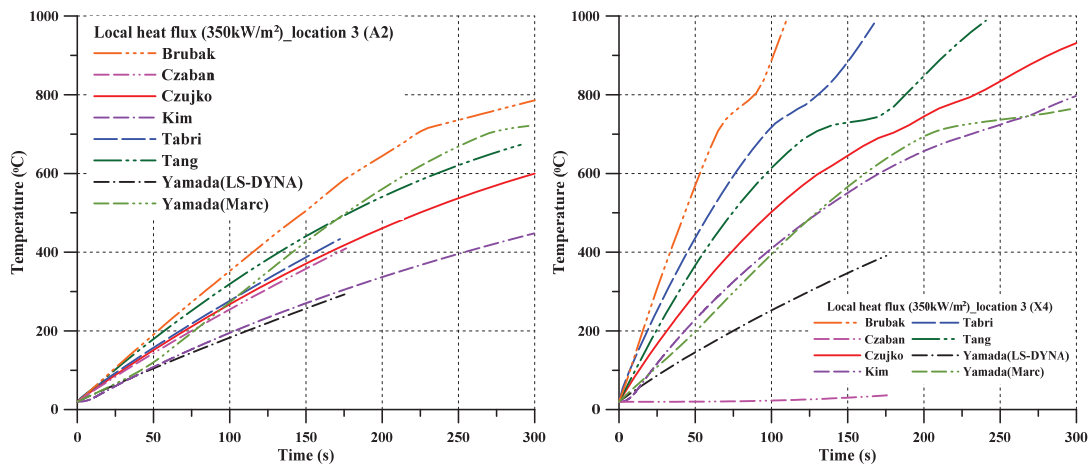
Figure 58. Comparison of temperature for local heat flux in location 3 at monitoring point A2 and X4.
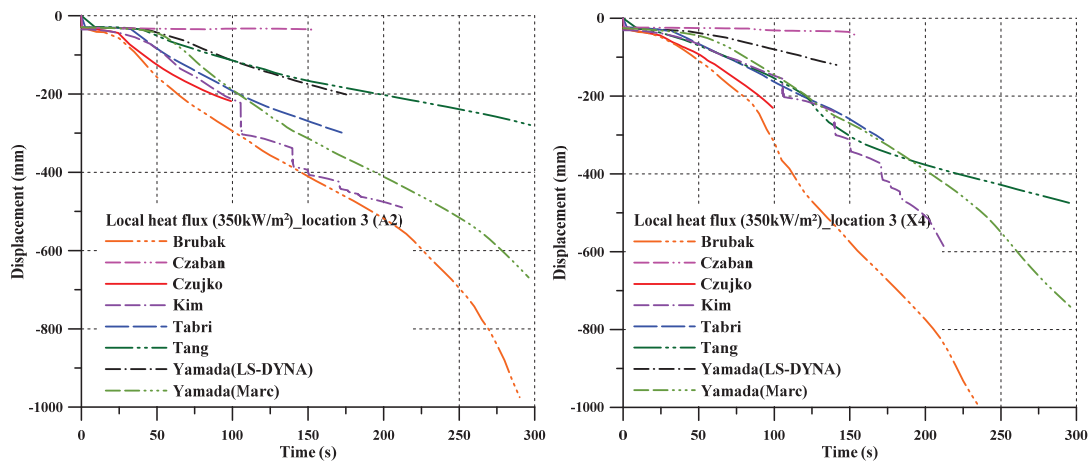
Figure 59. Comparison of deflection for local heat flux in location 3 at monitoring point A2 and X4.

7.4.7.4   *Local heat flux (location 4)*

This load case considers both global and local heat flux on the target deck structure. The local heat flux (350kW/m^2) was applied only in location 4 and global heat flux (100kW/m^2) was applied in the rest of deck structure. Figure 60 and Figure 61 show the results of local heat flux (location 4) at monitoring point A2 and Y3.
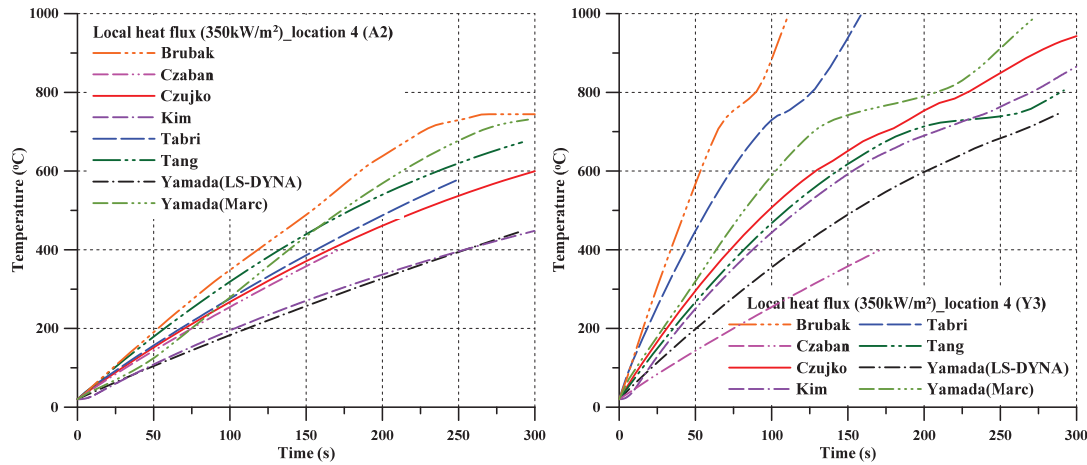
Figure 60. Comparison of temperature for local heat flux in location 4 at monitoring point A2 and Y3.
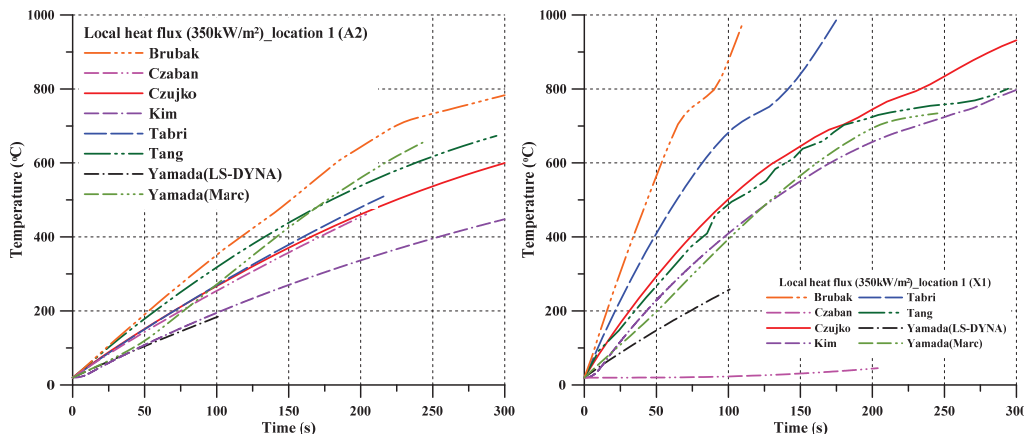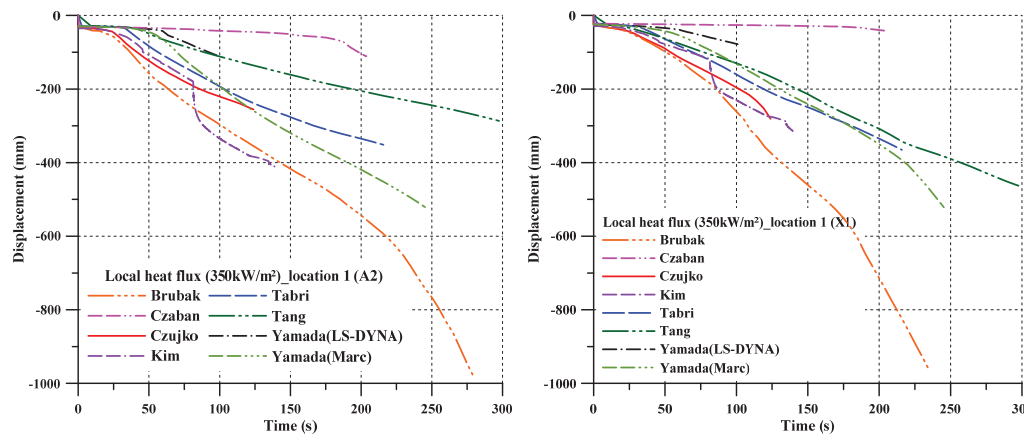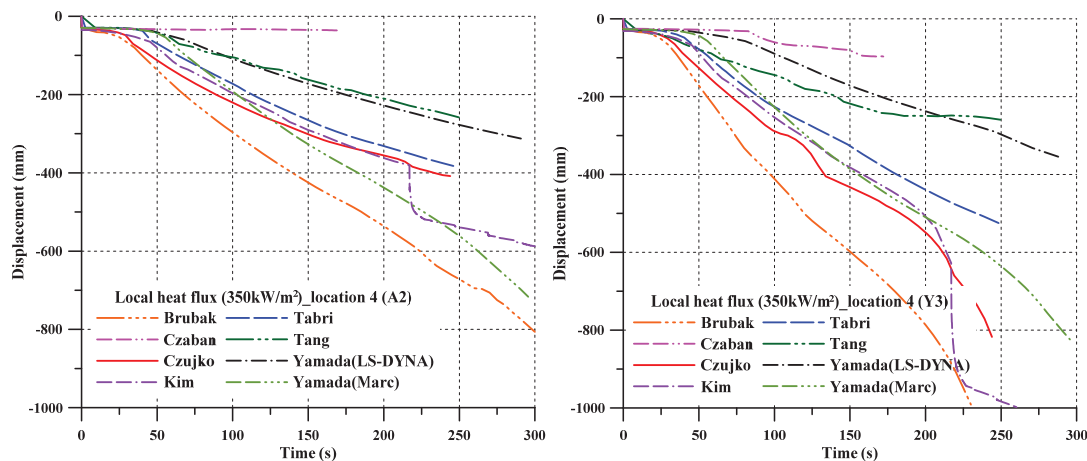

Figure 61. Comparison of deflection for local heat flux in location 4 at monitoring point A2 and Y3.

7.5   *Conclusion from the benchmark study*

The objective of the benchmark study is to predict the strength of topside structures subjected to fires and compare different techniques assessing the strength of these structures. The capabilities of modern software to simulate such complex loads are evaluated and the needs for Passive Fire Protection (PFP) design using numerical predictions are assessed.

The presented benchmark study consists of a relatively simple structural arrangement, i.e. a part of an offshore topside deck structure, subjected to fire loads. However, the study proved to be sufficiently complex enough to cause significant scatter in results when analyzed by a group of experts. This scatter is attributed to the underlying simulation assumptions made by the analysts. These results provide invaluable insight into the variability in predictions when different values are used for influential parameters, one of which is the analysts themselves.

The conclusions of primary study and parametric study are as follows:

Static analysis
This phase unveiled the influence of the individual approximations including the assumed in-place loading, geometric discretization, and boundary conditions. It was found that USFOS, ABAQUS and LS-DYNA were able to predict the static deflection with good accuracy.

Push-down analysis
To check the collapse strength of deck, push-down analysis was performed. The results indicate that the collapse strength of the deck model documented by each participant had good agreement with each other in the linear state, but had big differences (20%) in the plastic state.

Fire analysis
In the case of fire analysis, there were some differences. The temperature was different and as a result the deflection was also different. Therefore, in the case of fire analysis, thermal loading should be applied correctly in terms of the number of exposed sides, thermal properties and heat transfer. Design of PFP through numerical simulation was evaluated for beam and shell element model.

Effects of boundary conditions
The boundary conditions are an important factor for the proper calculation of the structural responses. The results indicate that very exact identification and modelling of boundary condition is necessary to correctly predict structural behavior in fire conditions.

Effects of local heat flux
To document the behavior of structure subjected to both global and local heat flux, present study was performed for 4 cases of local fires. The results indicated that local heat flux gives a larger damage than the constant/global heat flux.

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9.   ANNEX 1. MATERIAL MODELS FOR NON-LINEAR FINITE ELEMENT ANALYSIS

9.1   Introduction

Offshore structures exposed to hazards as defined above may undergo highly non-linear structural deformations, including rupture. Therefore, finite element analyses of these events require the input of appropriate material relations including failure representing the local material behaviour. Depending on the hazard to be analysed and the materials found on the offshore structures a selection of recommended material models can be made, see Table 13. The physical origin of these material models will be briefly presented, followed by numerical implementation possibilities as well as comments, hints and shortcomings arising from the use of those models as well as concerns of guidelines and standards. However,

Table 13. Recommended material models and associated hazards.

Hazard \ Material	Steel	Aluminium	Foam, Isolator, Rubber	Ice	Air	Water	Explosives	Risers, umbilical or power cables	Composite	Concrete	Seabed
Hydrocarbon explosions	■	■	■					■	■	■	
Hydrocarbon fires	■	■	■					■	■	■	
Underwater explosions	■	■	◆		■	■	■		◆	◆	◆
Wave Impact	■	■	■			■		◆	■		
Water-In-Deck	■	■	■			■			■		
Dropped Objects	■	■	■			◆		◆	■	■	■
Ship Impact	■	■	■	■				◆	■	■	◆
Earthquakes	■	■	■			◆		■	■	■	■
Ice, Iceberg	■	■	■	■		■		■	■	■	
Flooding	■	■	■		■	■			■		

■ - recommended, ◆ - recommended where applicable

hazard simulations utilizing the recommended material models and input parameters can be used for basic physical checks, but they may not be applicable in general.

The material modelling represents a crucial part of all numerical simulations, because it predefines how the material is assumed to behave during the simulations. Hence, the ability of the material model to represent the physical behaviour accurately directly influences the accuracy of the simulation results and their reliability. Furthermore, the correct physical behaviour may be represented well by the underlying assumptions of the material model, because it can correspond well to the physical experiment done to obtain the properties of the material in question. However, whether or not this experiment or the correspondence represents the true material behaviour remains often a question, e.g. a classical tensile experiment is a material test by agreement even though a structural test is carried out. Hence, the utilization of such experimentally based material models using small structural tests can lead to inconsistent results when applied to general structures. Furthermore, it remains often questionable whether the obtained material model corresponds to the discrete mathematical model, i.e. the finite element mesh, of the structure to be analysed. Hence, a material model should be unique and usable for any mesh size or conditions and should therefore not affect the results with a change in discretization of the simulation domain. In the past, often the term 'true' material model was utilized, which is however misleading as it implies that it is 'true' by all means and could be universally applied. In fact, all material measures are 'true' with respect to their determination scale, i.e. the engineering measure obtained by a tensile experiment is true with respect to the specimens' gauge length.

Hence, this chapter seeks to provide appropriate guidance to identify the material model to be used with the associated hazard according to Offshore structures exposed to hazards as defined above may undergo highly non-linear structural deformations, including rupture. Therefore, finite element analyses of these events require the input of appropriate material relations including failure representing the local material behaviour. Depending on the hazard to be analysed and the materials found on the offshore structures a selection of recommended material models can be made, see Table 13. The physical origin of these material models will be briefly presented, followed by numerical implementation possibilities as well as comments, hints and shortcomings arising from the use of those models as well as concerns of guidelines and standards. However, hazard simulations utilizing the recommended material models and input pa-rameters can be used for basic physical checks, but they may not be applicable in general.

Table 13 in such a way that it is consistent with the discretized, respectively meshed, simulation domain. Furthermore, engineering based best practices are provided as well as the associated shortcomings. The nomenclature of the numerical implementation used in the material input cards can be found in Hallquist (2007). The effects the material models account for, e.g. strain rate, temperature or damage criteria, will be provided alongside a selection of references relevant to the given material. Thereby, this database of material models will clarify common questions and uncertainties associated with the use of material models.

9.2   *Guidelines and standards*

ISO 19902 Ed 1 requires that the expected non-linear effects, including material yielding, buckling of structural components and pile failures, should be adequately modelled and captured. Strain rate effects should be considered as well as temperature dependency. NORSOK standard N-003 and DNV Recommended Practices DNV-RP-C204 suggest the use of the temperature dependent stress-strain relationships given in NS-ENV 1993 1-1, Part 1.2, Section 3.2. To account for the effect of residual stresses and lateral distortions compressive members should be modelled with an initial, sinusoidal imperfection with given amplitudes for elastic-perfectly plastic material and elastic-plastic material models. General class rules or CSR commonly state that an appropriate material model should be used; possibly in the form of a standard power law based material relation for large deformation analysis of steel structures. Additionally, some specify critical strain values to be used independent of the mesh size, which should, however, be sufficient, may be specified.

Hence, these guidelines and standards fail to provide a clear guidance for the analyst and may easily lead to diverse results simply by choosing different, yet not necessarily physically correct, material parameters.

9.3   *Material model database*

9.3.1   *Steel*

Commonly, the nonlinear material behaviour is selected in the form of a power law; see, for example, Alsos et al. (2009) and Ehlers et al. (2008). The power law parameters can be obtained from standard tensile experiments; see Paik (2007). However, with this approach agreement between the numerical

simulation and the tensile experiment can only be achieved by an iterative procedure for a selected element size chosen a priori. Hence, the procedure needs to be repeated if the element size is changed.

Furthermore, the determination of the material relation alone does not necessarily suffice, as the failure strain, i.e. the end point of the stress versus strain curve, depends in turn on the material relation. However, a significant amount of research has been conducted to describe criteria to determine the failure strain, for example by Törnqvist (2003), Scharrer et al. (2002), Alsos et al. (2008), and to present their applicability (e.g. Tabri et al. (2007) or Alsos et al. (2009). However, all of these papers use a standard or modified power law to describe the material behaviour, and none of these papers identifies a clear relation between the local strain and stress relation and the element length.

Relations to obtain an element length-dependent failure strain value are given by Peschmann (2001), Scharrer et al. (2002), Törnqvist (2003), Alsos et al. (2008) and Hogström et al. (2009). These curve-fitting relations, known as Barba's relations, are obtained on the basis of experimental measurements. However, they define only the end point of the standard or modified power law. Hence, Ehlers et al. (2008) conclude that the choice of an element length-dependent failure strain does not suffice in its present form.

Therefore, Ehlers and Varsta (2009) and Ehlers (2010b) presented a procedure to obtain the strain and stress relation of the materials, including failure with respect to the choice of element size using optical measurements. They introduced the strain reference length, which is a function of the discrete pixel recordings from the optical measurements and corresponds to the finite element length. As a result, they present an element length dependent material relation for NVA grade steel including failure, see Figure 9.1.

Moreover, Ehlers et al. (2012) identified that a constant strain failure criterion suffices for crashworthiness simulations of ship structures and that the strain rate sensitivity of the failure strain and ultimate tensile force is less than three per cent, see Figure 9.2. Hence, for moderate displacement speeds the strain rate influence is negligible.

An example input card following the LS-DYNA nomenclature for a piece wise linear material (mat_24) is given in Table 14.

Table 14. Piece wise linear steel material model.

```
*MAT_PIECEWISE_LINEAR_PLASTICITY
$#     mid        ro         e        pr      sigy      etan      fail      tdel
         1   7850.00  2.06E+11    0.3000   3.423E+8     0.000  0.661000     0.000
$#       c         p      lcss      lcsr        vp
     0.000     0.000
$#    eps1      eps2      eps3      eps4      eps5      eps6      eps7      eps8
     0.006   0.02612   0.04019   0.06865   0.15071     0.345   0.64477      0.74
$#     es1       es2       es3       es4       es5       es6       es7       es8
   3.423E+8  3.530E+8  3.731E+8  4.219E+8  4.901E+8  5.827E+8  6.621E+8  6.737E+8
```

However, the material behaviour, that is the change in the yield stress, at higher strain rates, $\dot{\varepsilon}$, can be calculated according to the Cowper-Symonds relation

$$1 + \left(\frac{\dot{\varepsilon}}{C}\right)^{1/p}$$

where C, p are the strain rate parameters and may be chosen as 40.4/sec and 5 for mild steel, respectively. Additionally, effects on elevated temperatures may be accounted for by scaling the global yield stress as a function of the temperature, see Figure 61. The increase in yield- and ultimate strength at cryogenic temperatures, i.e. -100 and -163 °C, is presented by Yoo et al. (2011) for mild stainless steel.
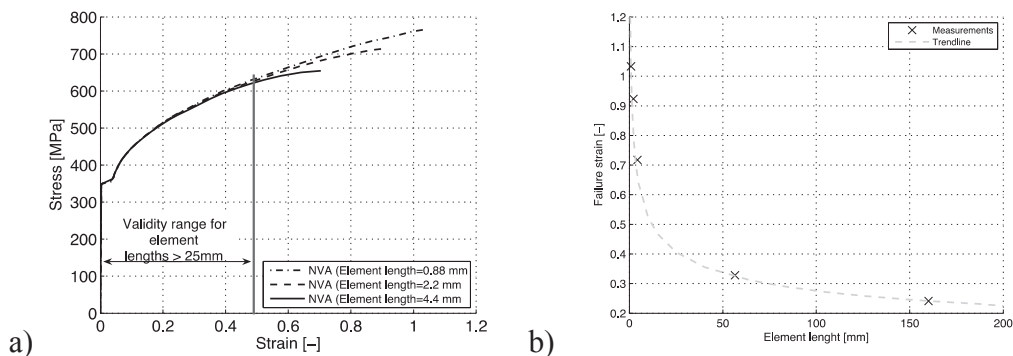

Figure 62. NVA grade steel: measured local strain and stress relation (a) and failure strain (b) (Ehlers, 2010a).
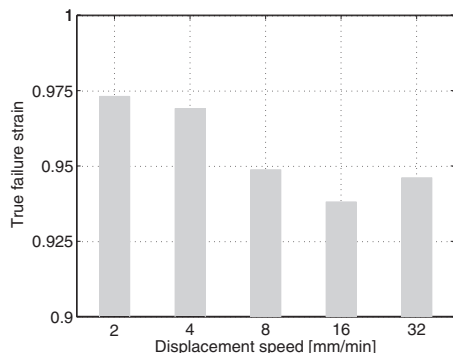
Figure 63. Influence of the displacement speed on the failure strain (Ehlers et al., 2012).
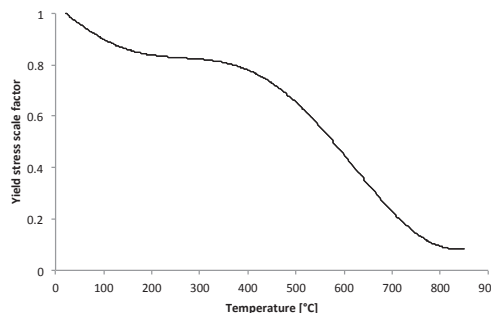

Figure 64. Global yield stress scale factor versus temperature for mild steel.

Definitial of thermal properties of materials requires additional keycards compared to the definition of basic material properties. A working example is presented in Table 15. *PART keyword should include the definitions for the basic material properties (marked with 355 in Table 9.3) and additional thermal material definition (marked with 2 in Table 15). Basic material definition is via *MAT_ELASTIC_PLASTIC_THERMAL, that defines gives the mechanical properties such as steel density and temperature dependent Young's moduli, Poisson's ratio, coefficients of thermal expansion, yield stresses and plastic hardening moduli. A maximum of eight temperatures with the corresponding data can be defined with a minimum of two points is needed. Keyword *MAT_THERMAL_ISOTROPIC_TD_LC allows additional isotropic thermal properties such as steel conductivity (tclc) and steel specific heat (hclc) to be specified by load curves. Finally, keyword *MAT_ADD_THERMAL_EXPANSION is used to apply the thermal expansion to a certain part according to a specified curve (curve no. 100 applied to the part no. 1 in Table 15). It should be noted that the latter overwrites the thermal expansion coefficients defined in *MAT_ELASTIC_PLASTIC_THERMAL keyword.

Table 15. Definition of thermal properties for steel (temperature in K).

```
*PART
Part 1 (Section 1, MAT 355, thermal material 2)
$#   pid    secid    mid    eosid    hgid    grav    adpopt    tmid
      1      1      355      0       0       0       0         2
*MAT_ELASTIC_PLASTIC_THERMAL
$#   mid      ro
     355  7.8500e-9
$#   t1     t2     t3     t4     t5     t6     t7     t8
     273    293    373    673    773    973    1073   1473
$#   e1     e2     e3     e4     e5     e6     e7     e8
    2.1e5  2.1e5  2.1e5  1.47e5 1.26e5 2.73e5 1.89e5  201
$#   pr1    pr2    pr3    pr4    pr5    pr6    pr7    pr8
     0.3    0.3    0.3    0.3    0.3    0.3    0.3    0.3
$# alpha1 alpha2 alpha3 alpha4 alpha5 alpha6 alpha7 alpha8
      0      0      0      0      0      0      0      0
$# sigy1  sigy2  sigy3  sigy4  sigy5  sigy6  sigy7  sigy8
     355    355    355    355    277    82     39     1
$# etan1  etan2  etan3  etan4  etan5  etan6  etan7  etan8
     200    200    200    200    200    200    200    200
*MAT_THERMAL_ISOTROPIC_TD_LC
$#   mid      tro     tgrlc    tgmult
      2    7.8500E-9
    hclc     tcl
      3       4
*DEFINE_CURVE_TITLE
Steel CONDUCTIVITY (TCLC), temperature in K
$#   lcid    sidr    sfa     sfo     offa    offo    dattyp
      4       0   1.0000  1.000000   273   0.000       0
0,54
```

```
  ....
1400,27.3
*DEFINE_CURVE_TITLE
Steel SPECIFIC HEAT (HCLC), temperature in K
$#   lcid    sidr     sfa     sfo    offa    offo  dattyp
     3      0  1.0000 1.000000    273   0.000      0
0,4.25E+08
  ....
1400,6.50E+08
*MAT_ADD_THERMAL_EXPANSION
1,100,1
*DEFINE_CURVE_TITLE
Steel THERMAL EXPANSION, temperature in K
$#   lcid    sidr     sfa     sfo    offa    offo  dattyp
    100      0  1.0000 1.000000    273   0.000      0
0,1.20E-05
  ....
1400,2.00E-05
```

9.3.2 Aluminium

Various thin-walled aluminium structures under crash behaviour, i.e. large deformations including rupture, have been analysed experimentally and numerically in the past.

Langseth et al. (1998) uses an elasto-plastic material model with isotropic plasticity following the von Mises yield criterion and associated flow rule, see Berstad et al. (1994). Strain rate effects are often neglected for aluminium alloys, such as AA6060, in the strain rate range of 10^4 to 10^3 s-1, see for example Lindholm et al. (1971). As a result, Langseth et al. (1998) are able to obtain good correspondence in terms of deformed shape, and shape of the force-displacement curve.

However, if high strain rates are to be expected, then the yield stress scaling according to Cowper-Symonds may be used. Nègre et al. (2004) study the crack extension in aluminium welds using the Gurson–Tvergaard–Needleman (GTN) model and obtain reasonable correspondence in terms of force versus crack mouth opening displacement (CMOD). However, the GTN model requires a vast amount of input parameters whose physical origin cannot be directly provided. Furthermore, Nègre et al. (2004) use 8-node brick elements, which are not suitable for large complex structures at present. Hence, from an engineering viewpoint this model does not suffice.

Lademo et al. (2005) utilize a coupled model of elasto-plasticity and ductile damage based on Lemaitre and Lippmann (1996) using the critical damage as an erosion criterion. They are able to simulate aluminium tensile experiments numerically with very good agreement using co-rotational shell elements and an anisotropic yield criterion Yld96 proposed by Barlat et al. (1997).

Such advanced material models can be easily implemented into numerical codes, and further increase in yield and ultimate strength at cryogenic temperatures, i.e. -100 and -163 °C, can be considered following the results by Yoo et al. (2011) for mild aluminium. Furthermore, a strain reference length-based approach using optical measurements as proposed by Ehlers (2010b) for steel may be used to obtain a consistent material relationship. However, for most analyses a consistent determination of the global material behaviour, see Figure, together with a Von Mises yield criterion will suffice.

An example input card following the LS-DYNA nomenclature for a piece wise linear material (mat_24) is given in Table 16.

Table 16. Piece wise linear aluminium material model

```
*MAT_PIECEWISE_LINEAR_PLASTICITY
$#     mid        ro         e        pr      sigy      etan      fail      tdel
        1   2.712E-9     75499    0.3000       200     0.000    0.1063     0.000
$#       c         p      lcss      lcsr        vp
     0.000     0.000
$#    eps1      eps2      eps3      eps4      eps5      eps6      eps7      eps8
   4.940E-4  8.928E-4  0.002087   0.01000   0.03630    0.0796
$#     es1       es2       es3       es4       es5       es6       es7       es8
    220.7480  230.8739  241.2000  253.2500  270.3999  293.2200
```
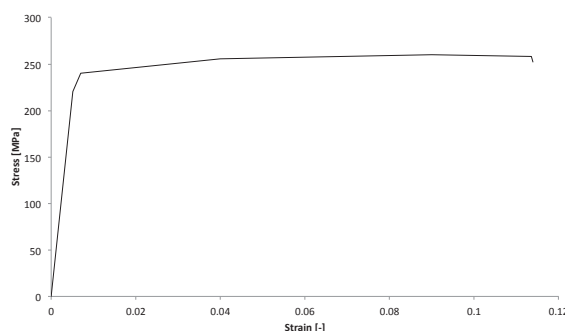
Figure 65. Example of a global strain versus stress curve from experiments.

9.3.3   Foam, Isolator, Rubber

Gielen (2008) presents an isotropic polyvinyl chloride (PVC) foam model, which exhibits elasto-damage behaviour under tension and elasto-plastic behaviour under compression. His damage model is consistent with the physical behaviour of the foam, a full-scale application and verification is however missing.

Cui et al. (2009) present a model for uniform foam based on Schraad and Harlow (2006) for disordered cellular materials under uni-axial compression. As a result, they obtain various influencing parameters affecting the energy absorption capacity under impact. Hence, functionally graded foams may be used to increase impact resistance.

In the case of rubber, a simplified rubber/foam material model (mat_181) may be used, which is defined by a single uni-axial load curve or by a family of uni-axial curves at discrete strain rates, see Figure 64. An example input card following the LS-DYNA nomenclature for such rubber material is given in Table 17.

Table 17. Simplified rubber/foam material model.

```
*MAT_SIMPLIFIED_RUBBER/FOAM
$#      mid         ro          k         mu          g       sigf        ref      prten
          1     1.75E-9       1000          0          0          0          0          0
$#      sgl         sw         st     lc/tbid     tension       eps6     avgopt    pr/beta
         80         50         15          1          0          0          0      0.495
```
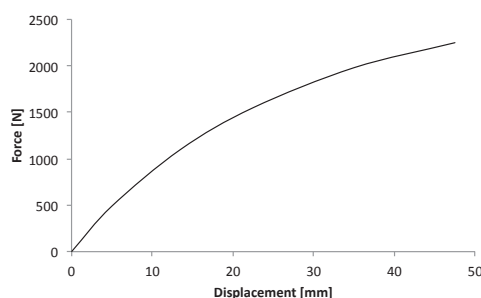

Figure 66. Exemplary force-displacement curve for rubber referenced as LC/TBID in mat_181.

9.3.4   Ice

One of the main difficulties when modelling ice is the prediction of ice failure, i.e. fracture, under loading at temperatures around the melting point of the ice. Thus the local ice-structure interaction includes transitions between the different phases. The failure process of ice begins when the edge of the moving ice hits the structure. This contact induces loads to the edge of the ice causing a stress state in the ice. When the stresses exceed the strength of ice, it fails. Ice becomes ductile with visco-elastic deformations during low loading rates and brittle during high loading rates.

Polojärvi and Tuhkuri (2009) developed specialized simulations tools utilizing the boundary element method, whereas Forsberg et al. (2010) utilizes the cohesive element method (CEM) to model ice failure. The latter is however of highly stochastic, or even random, nature and eventually results in reasonable agreement if experimental validation data becomes available.

However, Liu et al. (2011) treat the ice in a coupled dynamic ship – ice berg collision as an isotropic material, see Riska (1987), using the well-known Tsai-Wu strength criterion, see Tsai and Wu (1971). As

a result, the obtained numerical results give an indication of the structural damage of the ship structure. However, their model erodes the ice at failure in an unphysical fashion resulting in purely numerical pressure fluctuation in the contact surface.

Therefore, the underlying material models and ice properties are in need to be defined consistently to account for the possible scatter and thereby to result in reliable design methods for ships and offshore structures. Hence, unless material model data is not available explicitly for tension and compression including an appropriate failure criterion for brittle ice failure based on micro-crack growth, a simple elastic model may be employed. The latter is however only valid to some extent, if, e.g. the flexural strength of an ice sheet is of interest.

Therefore, as a first attempt, ice may be modelled as a volumetric body following non-iterative plasticity with a simple plastic strain failure model (mat_13). However, therein the yield- and failure stress is note rate or pressure dependent and the temperature is assumed constant. An example input card following the LS-DYNA nomenclature for Baltic Sea ice is given in Table 18.

Table 18. Simplified ice material model.

```
*MAT_ISOTROPIC_ELASTIC_FAILURE
$#     mid        ro         g      sigy      etan      bulk
        1 916.96000 3.0800E+9 7.6000E+5 6.8900E+9 8.0300E+9
$#     epf       prf       rem      trem
   0.010800 -3.080E+5     0.000     0.000
```

9.3.5 Air

For numerical simulations of structures subjected to underwater explosions, where the target is air-backed, the air needs to be modelled. The main material parameters are the mass density and the equation of state (EOS). The latter can be expressed as a linear polynomial defining the pressure in the gas as a linear relationship with the internal energy per initial volume. The ideal gas EOS is an alternative approach to the linear polynomial EOS with a slightly improved energy accounting algorithm. In most cases, the mass density is the only parameter defined for the air. The same material properties were used in Trevino (2000) and Webster (2007).

An example input card for air following the LS-DYNA nomenclature is given in Table 19 according to Webster (2007).

Table 19. Air material model.

```
*MAT_NULL (m, kg)
$#     mid        ro        pc        mu     terod     cerod        ym        pr
        1     1.280     0.000     0.000     0.000     0.000     0.000     0.000
```

The EOS example input following the LS_DYNA nomenclature is given in Table 20 according to Webster (2007) in the most common form, which defines the parameters such that it is an ideal gas behaviour.

Table 20. Linear polynomial equation of state for air.

```
*EOS_LINEAR_POLYNOMIAL (cm, g)
$#   eosid        c0        c1        c2        c3        c4        c5        c6
        1     0.000   1.0e-02     0.000     0.000     0.400     0.400     0.000
$#      e0        v0
     0.000     0.000
```

Do (2009) describes the calculation process of e0, which can be used to define an initial pressure within the air. Additionally, an example input card for the ideal gas EOS following the LS-DYNA nomenclature is given in Table 21 according to Martec-Limited (2007).

Table 21. Ideal gas equation of state for air.

```
*EOS_IDEAL_GAS
$#   eosid       cv0       cp0        cl        cq        t0        v0
        1  718.0000  1005.000     0.000     0.000    270.00     1.000
```

The ideal gas EOS is the equivalent of the linear polynomial with the C4 and C5 constants set to a value of $(\gamma - 1)$.

9.3.6   Water

When conducting simulations of structures subjected to underwater explosions, water models are required.

The primary mechanical property to be defined is the mass density and in some cases the pressure cut-off and dynamic viscosity coefficient is needed. The cut-off pressure is defined to allow the material to numerically cavitate when under tensile loading. This is usually defined as a very small negative number, which allows the material to cavitate once the pressure goes below this value.

Additionally, the equation of state (EOS) needs to be defined, most commonly as a Gruneisen EOS with cubic shock-velocity-particle velocity defining the pressure for compressed materials. The constants in the Gruneisen EOS are found from the shock wave velocity versus particle velocity curve. Two example input cards following the LS-DYNA nomenclature for water (mat_009) are given according to Trevino (2000) and Webster (2007) in table 22 and 23 respectively.

Table 22. Material model for water (Trevino, 2000).

```
*MAT_NULL (cm, g)
$#     mid        ro         pc         mu      terod      cerod         ym         pr
         1  1.000000      0.000      0.000      0.000      0.000      0.000      0.000
```

Table 23. Material model for water (Webster, 2007).

```
*MAT_NULL (m, kg)
$#     mid        ro         pc         mu      terod      cerod         ym         pr
         1  1025.000   -1.0e-20    1.13e-3      0.000      0.000      0.000      0.000
```

Additionally, Gruneisen EOS is the most commonly used EOS for defining the water behaviour with underwater explosion events. An example input card following the LS-DYNA nomenclature is given in Table 24 according to Webster (2007).

Table 24. Equation of state for water.

```
*EOS_GRUNEISEN
$#   eosid         c        s1         s2         s3      gamao          a         e0
         1  2417.000  1.410000      0.000      1.000      0.000      0.000      0.000
$#      v0
     1.000
```

9.3.7   Explosives

An explosive material requires two keywords to define the behaviour of the material. These include the material keyword and the equation of state (EOS). The mechanical properties to be considered are the mass density, the detonation velocity in the explosive and the Chapman-Jouguet pressure. Furthermore, the bulk modulus, shear modulus and yield stress may be required depending on the model.

For the EOS, there are three possibilities to define the pressure for the detonation products. All of these EOS define the pressure as a function of the relative volume and the internal energy per initial volume. The most commonly used EOS for explosive behaviour is the standard Jones-Wilkins-Lee (JWL). This EOS was modified by Baker and Stiel (1997) and has the added feature of better describing the high-pressure region above the Chapman-Jouguet state.

In addition to the material and EOS definitions in LS-DYNA, the INITIAL_DETONATION keyword is required to define the position and time of the initiation of the detonation process. This is the point at which the detonation initiates and the time for the remaining explosive to detonate is determined by the distance to the centre of the element divided by the detonation velocity. In the material definition for MAT_HIGH_EXPLOSIVE_BURN (mat_008) the value of BETA determines the type of detonation. If beta burn is used, any compression of the explosive material will cause detonation. For programmed burn, the explosive material can act as an elastic perfectly plastic material through the definition of the bulk modulus; shear modulus, and the yield stress. In this case, the explosive must be detonated with the INITIAL_DETONATION keyword.

An example input card following the LS-DYNA nomenclature for TNT (mat_008) is given in Table 25 according to Webster (2007).

Table 25. Explosive material model
```
*MAT_HIGH_EXPLOSIVE_BURN
$#    mid        ro         d       pcj      beta         k         g      sigy
       1  1630.000   6930.00    2.1e10     2.000     0.500     0.000     0.000
```

Furthermore, the most commonly used Jones-Wilkens-Lee EOS is given in Table 26 according to the LS-DYNA nomenclature (Webster, 2007).

Table 26. Equation of state for the explosive material model
```
*EOS_JWL
$#   eosid         a         b        r1        r2      omeg        e0        vo
       1   3.71e11    3.23e9      4.15     0.950     0.300     7.0e9     1.000
```

Keywords *LOAD_BLAST and *LOAD_BLAST_ENHANCED allow indirect modelling of the explosive and the propagation of blast wave without the need of actual discretization of the explosive or the air mesh around it. These keywords allow to define an airblast function for the application of pressure loads due to explosives described via equivalent mass of TNT. While *LOAD_BLAST only models the incident wave, the *LOAD_BLAST_ENHANCED includes enhancements for treating reflected waves, moving warheads and multiple blast sources. The loads are applied to facets defined with the keyword *LOAD_BLAST_SEGMENT. Example of indirect modelling is given in Table 27.

Table 27. Indirect modelling of explosive loading.

```
*LOAD_BLAST_ENHANCED
$#  bid    m    xbo    ybo    zbo    tbo    unit   blast
     1    30  -250000    0   6850  -0.53     5      2
$#  cfm    cfl    cft    cfp   nidbo  death  negphs
 2.205e3  3.28E-3   1e+3    145                  0
```

9.3.8   Risers, Umbilical or Power Cable

What all these structures have in common is the fact that they are typically very long, therefore slender. Their global mechanical properties to be defined are the bending-, torsional- and axial stiffness. Furthermore, the main aspect to be covered when modelling such structures is their stiffness dependency with respect to tension, torsion and curvature, i.e. stick-slip effects.

Therefore, experimental measurements of the global and local behaviour as well as a local analysis of the cross-section are needed. Typical numerical implementations would utilize elasto-plastic and visco-elastic material models considering friction, contact formulation (lift-off) as well as torsion/rolling effects on pipes.

Sævik (2011) studied the local behaviour of stresses in flexible pipes with a detailed model considering the cross-section build-up. However, for global analysis of an offshore structure, where the support effect of the slender structure is of interest, a simpler discretisation using beam elements with local stiffness properties can be used, see Rustad et al. (2008).

For a typical 8" flexible riser the following global parameters can be found: EI=200 kNm^2, EA=7.7 10^8 N, GI$_t$=5.9 10^6 Nm^2.

An example input card following the LS-DYNA nomenclature for a visco-elastic material (mat_117) is given in Table 28.

Table 28. Visco-elastic riser material model.
```
*MAT_VISCOELASTIC
$#    mid        ro      bulk        g0        gi      beta
       1  8650.000   2.06e11    0.8e11    0.1e11     0.200
```

9.3.9   Composites

Composite materials can be of various types, such as classical fibre-reinforced plastics or various stacks of materials, i.e. sandwich like structures. Therefore, their material parameters are very specific to the exact type of composite found in the offshore structure.

Menna et al. (2011) simulate impact tests of GFRP composite laminates using shells and provide the material parameters for a Mat Composite Failure Option Model (mat_059) of LS-DYNA. Feraboli et al.

(2011) present an enhanced composite material with damage (mat_054) for orthotropic composite tape laminates together with a series of material parameters.

Most orthotropic elastic materials can be described until failure according to:

$$\lfloor C \rfloor \{\sigma\} = \{\varepsilon\}$$

where C is the compliance matrix besides the six stress and strain components. Hence, the compliance matric can be composed of the extensional stiffness coefficients, the extensional-bending stiffness coefficients and the bending stiffness coefficients.

An example input card following the LS-DYNA nomenclature for a composite matrix material (mat_117) using such compliance matrix formulation is given in Table 29 for an equivalent stiffened plate.

Table 29. Composite material model.

```
*MAT_COMPOSITE_MATRIX
$#      mid        ro
          2 7850.0000
$#      c11        c12        c22        c13        c23        c33        c14        c24
   2.8409E+9 3.3956E+8 1.1319E+9     0.000      0.000 3.9615E+8 7.4958E+7 2.3769E+7
$#      c34        c44        c15        c25        c35        c45        c55        c16
       0.000 8.3506E+6 2.3769E+7 7.9231E+7     0.000 1.6645E+6 5.5485E+6     0.000
$#      c26        c36        c46        c56        c66       aopt
       0.000 2.7731E+7     0.000     0.000 1.9420E+6     0.000
$#       xp         yp         zp         a1         a2         a3
       0.000     0.000     0.000     0.000     0.000     0.000
$#       v1         v2         v3         d1         d2         d3       beta
       0.000     0.000     0.000     0.000     0.000     0.000     0.000
```

9.3.10 Concrete

Concrete material requires two keywords to define the behaviour of the material. These include the material keyword and the equation of state (EOS). The mechanical properties to be considered are the mass density, the shear modulus and an appropriate measure of the damage, respectively softening. The EOS describes the relation between the hydrostatic pressure and volume in the loading and unloading process of the concrete uncoupled from the deviatoric response. These parameters are typically obtained by experimental testing of the concrete under different loading directions and rates. Thus, the damage includes strain-rate effects.

Markovich et al. (2011) present a calibration model for a concrete damage model using EOS for tabulated compaction and a concrete damage, release 3, model (mat_72r3) and provide the required input parameters. Tai and Tang (2006) studied the dynamic behaviour of reinforced plates under normal impact using the Johnson–Holmquist Concrete equivalent strength model with damage and an EOS, which requires less input parameters and allows for easier implementation with good accuracy.

An example input card following the LS-DYNA nomenclature for concrete material (mat_111) is given in Table 30 according to Tai and Tang (2006).

Table 30. Concrete material model.

```
*MAT_JOHNSON_HOLMQUIST_CONCRETE
$#      mid        ro         g         a         b         c         n        fc
          1  2240.000 13.467e11     0.750     1.650     0.007     0.760     48.00
$#        t      eps0     efmin     sfmax        pc        uc        pl        ul
       0.000     1.000     0.010    11.700     13.60   0.00058     1.050     0.100
$#       d1        d2        k1        k2        k3        fs
       0.030     1.000     17.40     38.80     29.80     0.000
```

9.3.11 Soil

For some simulations of hazard the seabed has to be included. However, the material parameters for seabed, respectively soil, are fairly location dependent and may vary significantly within close proximities. Therefore, it is of utmost importance to obtain experimental data for the site in question.

Typically those experiments should identify the soil stiffness in different directions, the friction, the break out resistance and a cycling behaviour (trenching). Henke (2011) presents numerical and experimental results for Niederfelder sand and uses a hypoplastic constitutive model, assuming cohesion

less linear elastic behaviour, to achieve good correspondence. Vermeer and Jassim (2011) use a SPH approach with an elastic-plastic Mohr-Coulomb model to simulate drop anchors and present the utilized material parameters. Furthermore, solid elements can be used to represent sandy soils or granular materials following the Mohr-Coulomb behaviour.

An example input card following the LS-DYNA nomenclature for a Mohr-Coulomb material (mat_173) is given in Table 31 according to the material parameters from Vermeer and Jassim (2011)Vermeer and Jassmin (2011).

Table 31. Soil material model.

```
*MAT_MOHR_COULOMB
$#     mid        ro      gmod       rnu                phi      cval       psi
         1  1834.862    5.0e06     0.300         0     0.523    5.0e03     0.000
$#            nplanes              lccpdr     lccpt    lccjdr     lccjt    lcsfac
         0         0         0         0         0         0         0         0
$#  gmoddp    gmodgr    lcgmep    lcphiep   lcpsiep    lcgmst    cvalgr     aniso
     0.000     0.000     0.000     0.000     0.000     0.000     0.000  1.000000
$#     dip    dipang    cplane   frplane    tplane    shrmax     local
     0.000     0.000     0.000     0.000     0.000  1.00E+20     0.000
```

Another alternative for soil modelling is an isotropic material with damage that is available for solid elements. The model has a modified Mohr-Coulomb surface to determine the pressure dependent peak shear strength. It was developed for applications involving roadbase soils by Lewis (1999) for the Federal Highway Administration (FHWA), who extended the work of Abbo and Sloan (1995) to include excess pore water effects. Table 32 presents an example of FHWA soil model for compressed sand with the material properties obtained from Wang (2001) and FHWA (2004).

Table 32. Isotropic soil material model with damage.

```
*MAT_FHWA_SOIL
$#   mid      ro   nplot  spgrav  rhowat     vn  gammar  intrmx
      2  2.35e-9      1    2.65    1e-9    1.1      0      4
$#     k      g  phimax    ahyp     coh  eccen     an     et
     19     11   0.524  5.37e-4  6.2e-3    0.7      0      0
$# mcont    pwd1   pwksk    pwd2  phires   dint    vdfm  damlev
   0.034      0      0       0    1e-3  0.00001   6e-5   0.99
$# epsmax
      2
```

9.4 References

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