Proceedings of the 3rd Offshore Structural Reliability Conference (OSRC2016)

14-16 September 2016, Stavanger, Norway
Edited by Torgeir Moan and Amir R. Nejad

NTNU
Norwegian University of Science and Technology

Proceedings of OSRC2016

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14-16 September 2016
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OSRC2016 was organized under the auspices of International Association for Oil and Gas Producers (IOGP)

Conference sponsors:

Preface

The goal of the OSRC conferences is to serve as a place for knowledge exchange relating to the life cycle structural integrity management of offshore structures. Inspired by the workshop organized by Richard Snell in BP in 1995, the 1st OSRC conference was organized under the auspices of IOGP at BP in London in 2012 and chaired by Phil Smedley. The 2nd conference was organised by API in Houston in 2014 and chaired by Dave Wisch, Chevron. It has been decided to hold the conference biennially - independent of the oil price.

The 3rd conference was held in Stavanger on September 14-16 2016. All speakers were invited with the intention to cover various topics such as airgap, freeboard and wave loads in deck, accidental events and risk assessment, stability and station-keeping requirements to floating structures, aging structures – reassessment, inspection, monitoring, maintenance and repair, robustness, uncertainty assessment and use of reliability and risk assessment methodologies. A particular focus was on principles and procedures relevant for the development of standards, such as ISO 19900, for the integrity management of oil and gas platforms.

The PowerPoint presentations and accompanying papers for 14 presentations were made available to the attendees before the conference. The report of the discussion sessions were presented to the participants, after an iterative quality assurance and control, in early January 2017.

By allowing contributors to consider submitting their papers to Journal of Marine Structures, the journal conducted a review process of submitted papers until the fall of 2017. Five papers will be published in the journal. The preprints of these papers are included in these proceedings. So all together these proceedings include

- the conference program
- 38 PowerPoint presentations, including 3 keynotes
- 10 Papers and preprints of 5 papers to the Journal of Marine Structures
- a report of the discussion at the conference

organized according to sessions.

The 3rd OSRC was organized in close cooperation with the ISO and IOGP committees on structures, chaired by Phil Smedley (BP) and Simen Moxnes (Statoil), respectively. We appreciate the financial support from DNVGL, IOGP, NTNU and Statoil, and the cooperation of Alf Reidar Johansen (IOGP), Suzanne Lacasse (NGI), Henrik O. Madsen (DNVGL) and Simen Moxnes (Statoil) as well as the international advisory committee in organizing this conference.

Trondheim, 20 March 2018

Torgeir Moan
Amir R. Nejad

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Conference program

The 3rd Offshore Structural Reliability Conference

OSRC2016

14–16 September, Stavanger, Norway

Welcome to the 3rd OSRC

Oseberg Field Centre - Photo: Statoil

PROGRAM

DNV·GL

International Association of Oil & Gas Producers

NTNU – Trondheim
Norwegian University of Science and Technology

Statoil

The 3rd Offshore Structural Reliability Conference (OSRC) takes place in Stavanger on September 14-16 2016. The venue is Statoil Business Centre, Stavanger, Norway. This event is organized under the auspices of International Association for Oil and Gas Producers (IOGP); with a strong link to the International Organization for Standardization (ISO) in connection with the development of standards for offshore structures. The event will hence gather major stakeholders from the offshore oil and gas industry to discuss safety issues of importance to the industry and society for a safe and sustainable exploitation of hydrocarbons. Three keynote lectures and about 30 other invited lectures will be delivered as a basis for exchange of opinions on important issues such as service experiences, target safety level, extension of service life of existing platforms, floating platforms, the need for safe yet cost efficient technological solutions and standards.

We would like to welcome you to three interesting and rewarding days in Stavanger.

Time and place

When:
14–16 September 2016

Where:
Statoil Business Centre, Stavanger, Norway
Address: Forusbeen 50, 4035 Stavanger

Organising Committee

Alf Reidar Johansen, IOGP
Suzanne Lacasse, NGI
Henrik O. Madsen, DNVGL
Torgeir Moan, NTNU (chair)
Simen Moxnes, Statoil

Conference program – overview

DAY	TIME	SESSION	TOPIC	CO-CHAIRS
Wednesday 14.09.2016	0900-1000	Opening & Keynote No. 1	Opening What Characterizes a Reliable Structure?	T. Moan; A. R. Johansen
	1000-1110	S1	Metocean Conditions	P. Tromans; O. T. Gudmestad
	1110-1130	Break		
	1130-1220	S2	Wave Environment and Loads	C. Swan; S. Haver
	1220-1310	S3	Reliability of Jackets in Severe Wave Conditions	M. Birades; J. Waegter
	1310-1400	Lunch		
	1400-1450	S4	Reliability based Calibration of ULS Code Criteria	H. O. Madsen; A. Mangiavacchi
	1450-1600	S5	Accidental Collapse Limit States	Y. S. Choo T. Sildnes
	1600-1620	Break		
	1620-1730	S6	Reliability of Concrete Platforms	J. Moksnes C. O'Brien
	1800	Reception		

DAY	TIME	SESSION	TOPIC	CO-CHAIRS
Thursday 15.09.2016	0900-0940	Keynote No. 2	Should Global Standards Specify Reliability Values for Design and Assessment?	S. Moxnes; T. Moan
	0940-1030	S7	Reassessment of Jacket Platforms in Operation	F. Nadim; J-L. Colliat-Dangus
	1030-1050	Break		
	1050-1200	S8	Inspection Planning with respect to Crack Control	H. O. Madsen; P. Frieze
	1200-1250	S9	Reliability of Mobile Units	O. Dalane; C. Wang;
	1250-1340	Lunch		
	1340-1430	S10	Stability of Floating Platforms in a Reliability Perspective	J. Stear; T. Moan
	1430-1520	S11	Reliability of Floating Platforms	A. van der Stap; T. Vestbøstad
	1520-1540	Break		
	1540-1630	S12	Reliability of Ship type Production Units	T. Sildnes; C. Wang
	1630-1720	S13	Reliability of Station-keeping Systems	P. Smedley; E. Hovland
	1900	Dinner		

DAY	TIME	SESSION	TOPIC	CO-CHAIRS
Friday 16.09.2016	0900-0940	Keynote No. 3	API 2GEN – Overarching Document for Developers of the API Series 2 Standards	S. Lacasse; T. Moan
	0940-1030	S14	Floating Arctic Structures	P. Liferov; M. Maes
	1030-1045	Break		
	1045-1255	S15	ISO 19900 – Presentations & Discussions	M. Maes; H.O. Madsen
	1255-1300	Closing	Closing Remarks	T. Moan
	1300-1345	Lunch		

Keynote Lectures

➲ Keynote No. 1:
What Characterizes a Reliable Structure?
Simen Moxnes, Statoil, Norway

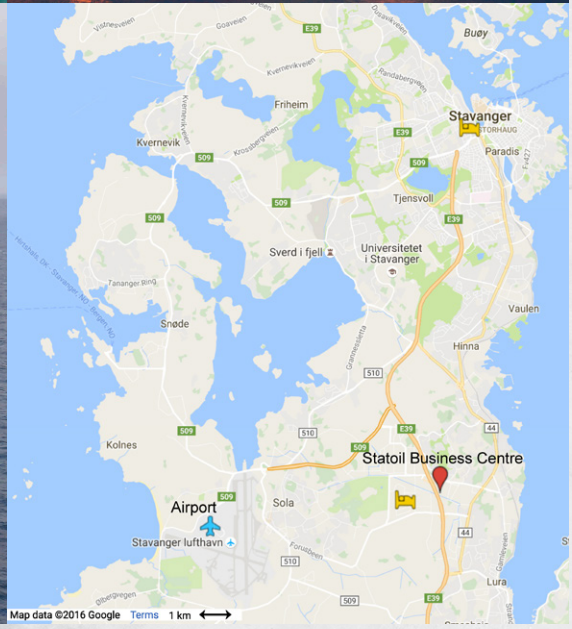
➲ Keynote No. 2:
Should Global Standards Specify Reliability Values for Design and Assessment?
Philip Smedley, BP, UK

➲ Keynote No. 3:
API 2GEN – Overarching Document for Developers of the API Series 2 Standards
Dave Wisch, Chevron, USA


Sleipner A platform storm - Photo: Øyvind Hagen - Statoil


Arctic Princess left port at Melkøya - Photo: Allan Klo - Statoil


Visund - Photo: Kjetil Alsvik - Statoil



Sessions

➲ Session 1: Metocean Conditions

Co-chairs: P. Tromans and O.T. Gudmestad

Waves and Associated Current – Experiences from a Five Year Measurement Campaign in the northern North Sea
Kjersti Bruserud, Statoil and Sverre Haver, UIS/NTNU, Norway

Airgap and Safety: Metocean Induced Uncertainties Affecting Airgap Assessments
Sverre Haver, UiS/NTNU, Norway

Wave Kinematics and Hydrodynamic Loads on the Tyra Jacket Inferred from Systematic Model Testing and Field Measurements
Jesper Tychsen, Maersk Oil, Denmark

➲ Session 2: Wave Environment and Loads

Co-chairs: C. Swan; S. Haver

Demonstrating Draugen ALS/ULS Compliance despite Significant Wave/Ringing Load Increases since Original Design
Guido Kuiper, Norske Shell, Norway

Slamming Loads from Steep and Breaking Waves
Gunnar Lian and Tone Vestbøstad, Statoil, Norway

➲ Session 3: Reliability of Jackets in Severe Wave Conditions

Co-chairs: M. Birades; J. Waegter

Summary of the Impact on Reliability by the Tyra Field Extreme Wave Study 2013-15
Jesper Tychsen, Maersk Oil et al., Denmark

The Loads JIP: The Loading and Reliability of Fixed Steel Structures in Extreme Seas
Chris Swan, Imperial College

➲ Session 4: Reliability Based Calibration of ULS Code Criteria

Co-chairs: H.O. Madsen; A. Mangiavacchi

Risk-Based Codification of Structural Design and Assessment: Benefits and Challenges
Marc Maes, University of Calgary, Canada

Uncertainty Assessment of Geotechnical Design and Calibration of Resistance Factors for Offshore Piles
Farrokh Nadim, NGI, Norway

➲ Session 5: Accidental Collapse Limit State

Co-chairs: Y.S. Choo; T. Sildnes

Standards – Targets and not Risk Management
Dave Wisch et al., Chevron, USA

Assessment of Ship Collision Risk in the North Sea: Recent Guidelines
Gerhard Ersdal, PSA, Norway, et al.

Non linear, Dynamic Analysis of Long term Blast Loading on a Topsides Compression Skid
Himanshu Singh, Shell Global Solutions, the Netherlands

➲ Session 6: Reliability of Concrete Platforms

Co-chairs: J. Moksnes; C. O'Brien

Reliability of a Concrete Floating Barge – the NKP Case
Pascal Collet, Total, France

Experiences with the Safety and Durability of Concrete Offshore Platforms
Kolbjørn Høyland, Olav Olsen AS, Norway

ALARP in Decommissioning Brent D
Frank Lange, Shell Global Solutions

➲ Session 7: Reassessment of Jacket Platforms in Operation

Co-chairs: F. Nadim; J-L. Colliat-Dangus

Risk-based Structural Integrity Management for Jacket Structures
Francis Guede, Bureau Véritas, France

Reassessment of Offshore Structures: the Geotechnical Issues
Thomas Langford, NGI, Norway, et al.

➲ Session 8: Inspection Planning with Respect to Crack Control

Co-chairs: H.O. Madsen; P. Frieze

Lessons Learned from Predicted Versus Observed Fatigue of Offshore Steel Structures (Jackets, Semis) in the North Sea
Ole Tom Vårdal, Axess AS and T. Moan, NTNU, Norway

Guidelines for Probabilistic Inspection Planning of Offshore Steel Structures
Gudfinnur Sigurdsson, DNVGL, Norway

Fatigue Analysis, Lifetime Extension and Inspection Plans
Michel Birades, Total and Laurent Verney, Bureau Véritas, France

➲ Session 9: Reliability of Mobile Units

Co-chairs: O. Dalane, C. Wang

Operational Experiences and Design Codes for MODU
Tore Sildnes, DNVGL, Norway

Reliability of Jack-up Platforms
Mike Hoyle, DNVGL

➲ Session 10: Stability of Floating Platforms in a Reliability Perspective

Co-chairs: J. Stear; T. Moan

Assessment of Intact and Damage Stability Regulations for Offshore Floating Structures – in a Reliability and Risk Perspective
Christina Wang, ABS

Reliability of Floating Platforms with respect to Stability
Dag Erling Engberg, DNV GL, Norway

➲ Session 11: Reliability of Floating Platforms

Co-chairs: A. van der Stap; T. Vestbøstad

Industry Standards for Integrity Management of Floating Systems
Jim Stear, Chevron, USA

ALS Design in Practice – Floating Platform Application
Rolf Løken, Aker Solutions, Norway

➲ Session 12: Reliability of Ship type Production Units

Co-chairs: T. Sildnes; C. Wang

FPSOs in Harsh Environment. Status and Main Learnings after 30 years' Experience with One of Our Most Robust, Reliable and Flexible Concepts
Erlend Hovland, Statoil, Norway

Design Basis of World's First FLNG to Achieve Good Reliability
Andre van der Stap, Shell Global Solutions, the Netherlands

➲ Session 13: Reliability of Station-keeping Systems

Co-chairs: P. Smedley; E. Hovland

NorMoor JIP – Mooring Design Code Calibration
Siril Okkenhaug, DNVGL, Norway

Reliability of DP Systems
Haibo Chen, Lloyds Register, China

➲ Session 14: Floating Arctic Structures

Co-chairs: P. Liferov; M. Maes

Sea Ice Management and Reliability of Floating Structures
Richard McKenna, McKenna & Associates, and Brian Wright, Canada

Applying the Limit State Definition in Ice Class Rules for Ships to Offshore Structures
Kaj Riska and R. Bridges, Total, France

➲ Session 15 - on ISO 19900 WG1 ongoing work

Co-chairs: M. Maes and H.O. Madsen

Background: Developing a third edition of ISO 19900 by WG1 (M. Maes, M. Abraham, S. Balasubramanian, M. Birades, P. Frieze, M. Hoyle, K. Høyland, A. Mangiavacci , T. Moan, S. Moxnes, P. Smedley, J. Waegter, D. Wisch)

The session consists of presentations and discussion of the preliminary results of the work of technical panels on:

- Limit States & System Effects
  (T. Moan -lead), (ALS/ULS Clarification, Accidental vs. Abnormal Actions, Damaged Members, Robustness and System Effects, Non-structural Robustness, Representative vs. Characteristic Actions and Resistances)
- Risk, Consequence, and Reliability Classes/Targets
  (P. Smedley -lead)(Exposure Levels, Consequence Classes, Quality Control, Reliability Targets)
- Uncertainty Assessment
  (P. Frieze-lead) (Actions, Action Effects, Resistances – also Geotechnical – and Impact on ULS and FLS Requirements
- Lifetime Extension
  (S. Moxnes –lead)

Sword in rock Hafrsfjord – Photo: Richard Larssen/visitnorway.com

Chapter 2

Keynotes & opening session

2.1 Opening by Torgeir Moan

NTNU
Norwegian University of Science and Technology

International Association of Oil & Gas Producers

WELCOME to OSRC 2016

The 3rd Offshore Structural Reliability Conference
OSRC2016
14-16 September, Stavanger, Norway

Opening Address

by
Torgeir Moan

Good morning everyone and
welcome to Stavanger and the third OSRC.

➢ The Subject area

Reliability in a broad sense ; defined as follows by ISO 2394;
- The ability of a structure or structural member to fulfil the specified requirements, including the working life, for which it has been designed.
- Reliability is often expressed in terms of probability.
- NOTE:
  Reliability covers safety, serviceability and durability of a structure.

➢ Background

- 1st conference under the auspices of IOGP at BP in London in 2012
  Chair: Philip Smedley
  (inspired by the 1995 BP workshop; organized by Richard Snell);
- 2nd conference was organised by API in Houston in 2014.
  Chair: Dave Wisch

- The conference is to be held biennally - independent of oil price

The reliability perspective

- Overall experiences: the reliability depends the human factors in the life cycle of structures
- Past experiences in different industry sectors is that safety is at stake during periods with
  - economic pressure (recession, downsizing, …)
  - time pressure and
  - introduction of new technologies
    (advanced computational tools, without deep insight..)
- The main measure to ensure safe operations is that everybody involved have a
  - safety attitude and adequate competence
  - and that knowledge – i.e. best practice and lessons learnt - is properly transferred to enable addressing future challenges – especially in times with economic and time pressure,

We hope that this conference can serve as a place for knowledge exchange relating to safe offshore operations in the future - towards **best practice**.

Conference scope

We have **invited speakers to highlight particular issues relating to**

- airgap, freeboard and wave loads in deck
- accidental events and risk assessment
- stability and station-keeping requirements to floating structures
- aging structures – reassessment, inspection, monitoring, maintenance, repair – steel and reinforced concrete struct.
- matters of interest for the revision of ISO 19900 (limit states, robustness, uncertainty, service life extension,
- target reliability
  - how safe is safe enough – in a societal and ALARP perspective : principles and
  - practices in different regions
  - for new versus existing structures
  - continental shelf versus maritime regulatory regimes – i.e. ISO versus IMO
- Presentation 3 in Session 6 is unfortunately withdrawn

Discussion

- Discussion after the presentation in each session
- Discussions are important for the future work with the ISO and other standards – and are encouraged.
- Discussions will be *reported to IOGP* and anyone interested.
  - Amir, Asle and Chenyu will make notes
  - we would appreciate that discussers and speakers provide their comments/questions and response, respectively, in writing - to Amir, Asle or Chenyu (deadline for discussers: by email Sept. 30. 2016)
  - co-chairs keep track of discussers

Practical information

- Co-chairs should keep the time – to allow for QA
- All of you
  - can download presentations (until mid - Oct.)
  - should provide discussions (also) in writing by mid - Oct.
- Internet : register on a particular form (by help of a Statoil employee)
- Internet café : on this floor
- Transport
- Lunches every day: on the first floor
- Reception today: on the first floor
- Dinner tomorrow in the Stavanger Concert Hall (with bus transport to downtown)

Acknowledgement

- The close cooperation with ISO and IOGP committees on structures, chaired by Philip Smedley and Simen Moxnes, respectively
- DNVGL, IOGP, NTNU and Statoil for sponsoring the conference
- Alf Reidar Johansen, Suzanne Lacasse, Henrik O. Madsen and Simen Moxnes – and Amir Nejad for their cooperation in organizing this conference.

Finally

- Again very welcome.
- May we all gain by it.
- May we all have a good time.
- Thank you for coming.

2.2 Keynote 1 by Simen Moxnes

NTNU Norwegian University of Science and Technology

International Association of Oil & Gas Producers

What characterizes a reliable structure?

Simen Moxnes
Chief Engineer Platform Technology
Statoil ASA

The 3rd Offshore Structural Reliability Conference
OSRC2016
14-16 September, Stavanger, Norway

2016 Offshore Structural Reliability Conference

The 1995 Structural Reliability Workshop

FIXED STEEL	CONCRETE
COMPONENTS 4 x 10^{-4} FOUNDATION (C) 10^{-3} SYSTEM 10^{-4} INCL. JOINTS	COMPONENT REBAR 10^{-4} - 10^{-5} CONC 10^{-5} - 10^{-6} FOUNDATION $\approx 10^{-4}$ SYSTEM -10^{-2} ON STRUCTURE BUT ? ON FOUNDATION FLOATER WATERTIGHTNESS EXCL. JTS & SHEAR
SHIPS	**SEMI & TLP**
HULL GIRDER 10^{-3} - 10^{-5} STABILITY ? * SUPPLEMENT TO PROD CLASS	TENDONS 10^{-4} - 10^{-5} HULL 10^{-3} - 10^{-4} DECK AS FIXED PLATFORM *MOORINGS CHAIN 10^{-2} WIRE 2.3 10^{-3} *AIR GAP

* ISO ACTION

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Offshore Structures

Fixed plattforms

Jacket 

Jack-up 

Gravity base 

Compliant tower 

Floating production units

Semi 

Spar 

TLP 

Floating production and storage

Ship shape 

Barge type 

Circular buoy 

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Structural reliability –
ISO2394 General principles on reliability for structures

2.1.8
reliability

ability of a structure or structural member to fulfil the specified requirements, during the working life, for which it has been designed.

Structural reliability –
ISO2394 General principles on reliability for structures

2.1.8
reliability
ability of a structure or structural member to fulfil the specified requirements, during the working life, for which it has been designed.
Note 1 to entry: Reliability is often expressed in terms of probability.
Note 2 to entry: Reliability covers safety, serviceability, and durability of a structure.

2.1.20
member reliability
reliability of a single structural member which has one single dominating failure mode

2.1.21
system reliability
reliability of a system of more than one relevant structural member or a structural member which has more than one relevant failure mode

Structural reliability – ISO 19900

3.48
structural reliability analysis
procedure for the determination of the level of safety against failure of a structure or structural component

Structural reliability – ISO 19900

3.48
structural reliability ~~analysis~~
~~procedure for the determination of~~ the level of safety against failure of a structure or structural component

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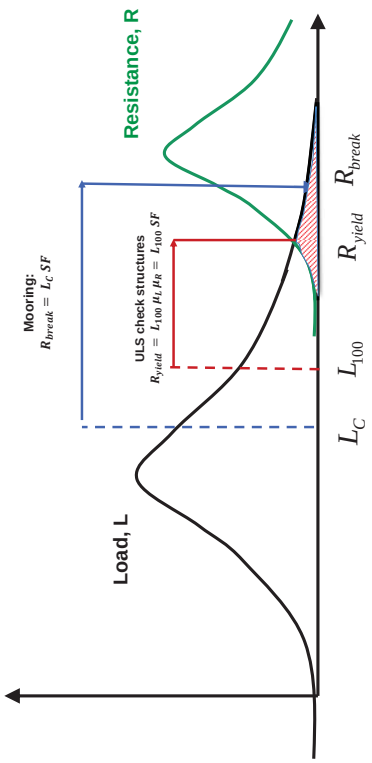
Reliability = Safety Factor?



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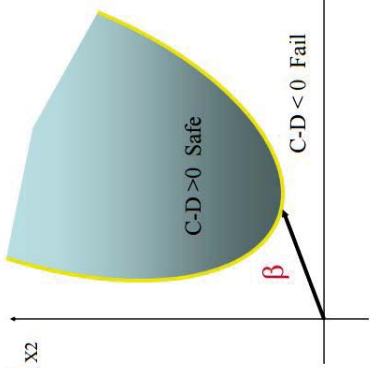
Reliability = Reliability index ?



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Stability of floating structures

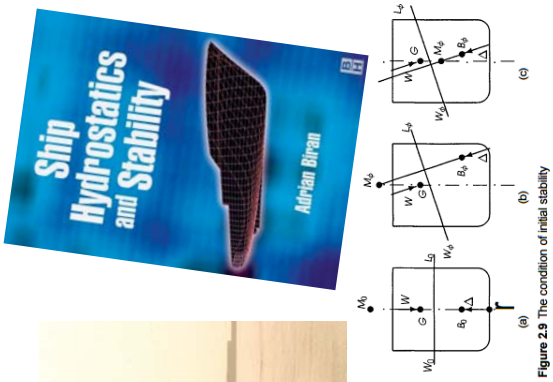

Figure 2.9 The condition of initial stability

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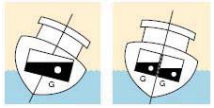
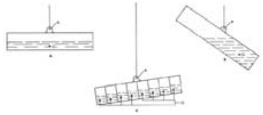
MS Herald of Free Enterprise - 1987





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Alexander Kielland accident

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Robustness – ISO 19900

3.42

robustness

ability of a structure to withstand accidental and abnormal events without being damaged to an extent disproportionate to the cause

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Robustness – some important aspects

- Utilisation
- Ductile failure modes
- Redundancy
- Ability to redistribute loads

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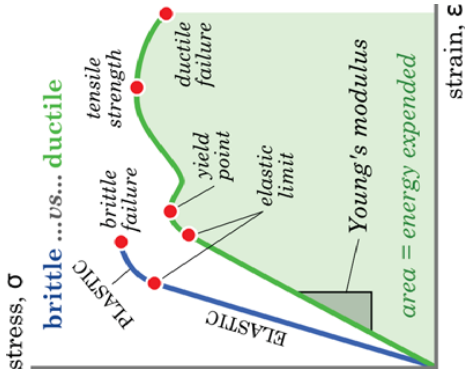
Ductile vs brittle behaviour



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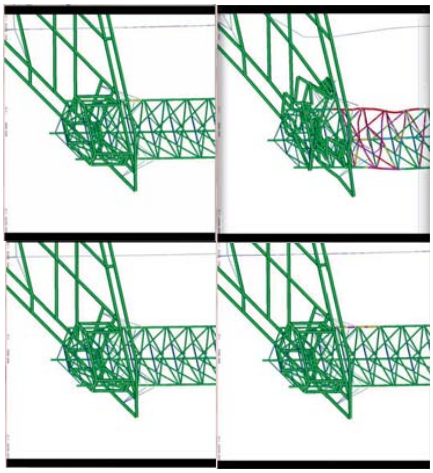
Push over analysis – 8 legged jacket



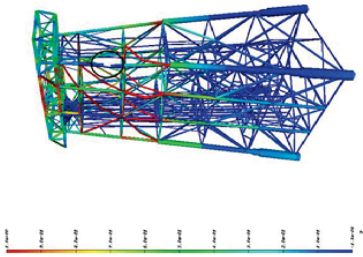


Figure 20 Failure mode, Corner leg damaged

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Robustness – Reserve buoyancy

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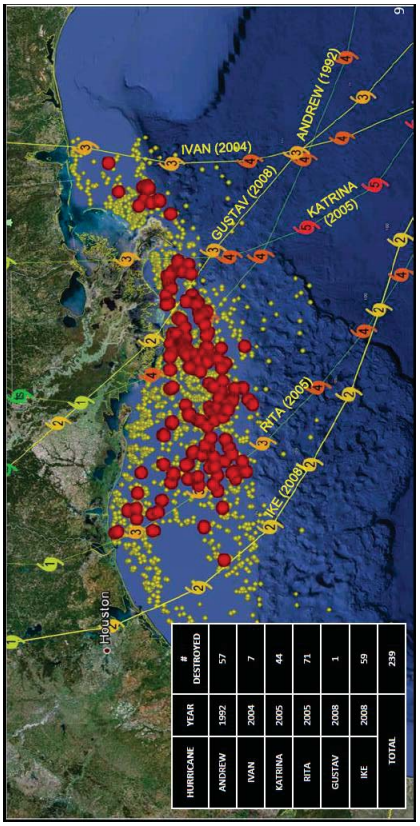
Significant structural failures in GoM Hurricanes (Puskar/Gilbert OSRC 2012)



HURRICANE	YEAR	# DESTROYED
ANDREW	1992	57
IVAN	2004	7
KATRINA	2005	44
RITA	2005	71
GUSTAV	2008	1
IKE	2008	59
TOTAL		239

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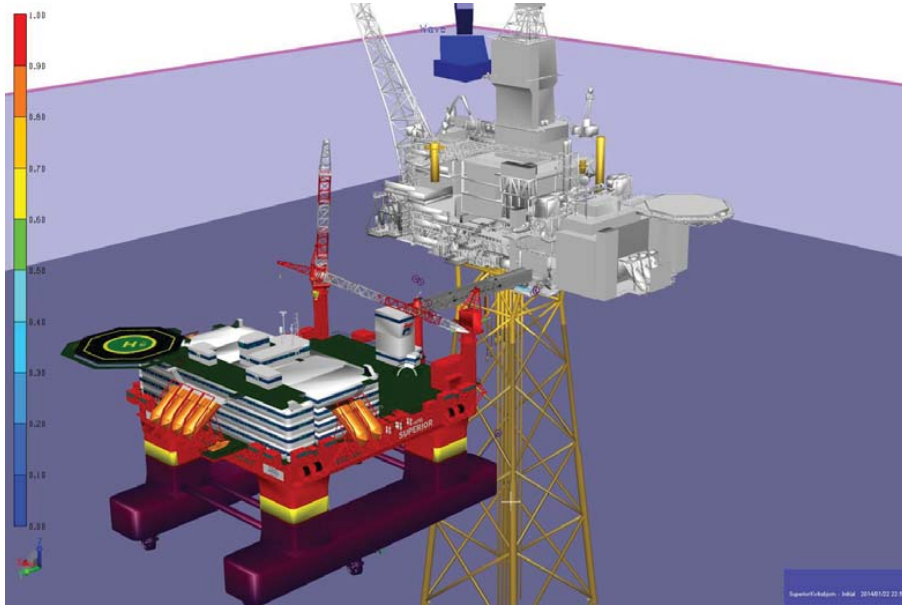
Safety in design – Non-structural issues



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Robustness – ISO 19900

5.3 Robustness

A structure design shall incorporate sufficient robustness to ensure that consequent damage is not disproportionate to the cause. For a robust structure, local damage does not lead to complete loss of integrity of the structure. Robustness can also ensure that structural integrity in a damaged state is sufficient to allow a process system shutdown, isolation of the reservoir and a safe evacuation where applicable

Robustness can be achieved

a) by ensuring (by design or by protective measures) that no critical component exposed to hazard can be made ineffective; or
b) by providing alternate load-carrying paths (structural redundancy) in such a way that any single load-bearing component exposed to a hazard can be made ineffective without causing collapse, sinking, or capsize of the structure or any significant part of it; or
c) by a combination of a) and b).

A floating structure shall incorporate sufficient damaged stability and reserve of buoyancy to ensure that credible scenarios of unintended flooding do not result in loss of the structure.

The stationkeeping systems of floating structures shall incorporate sufficient redundancy to ensure that the structure can withstand loss of a stationkeeping component (e.g. mooring line(s)) in accordance with the provisions of ISO 19901 7.

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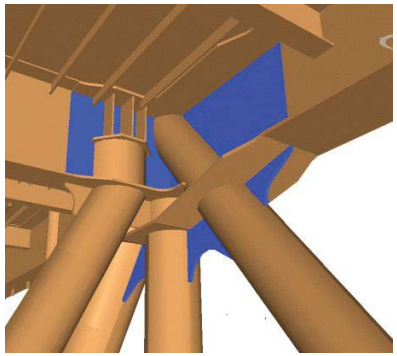
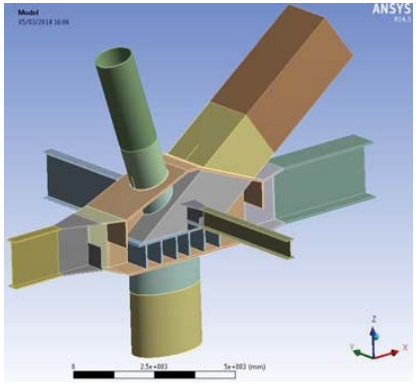
Is this a reliable structure?

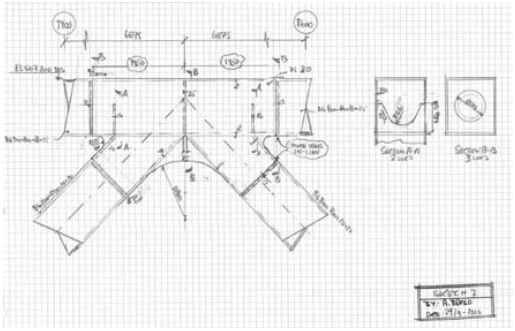


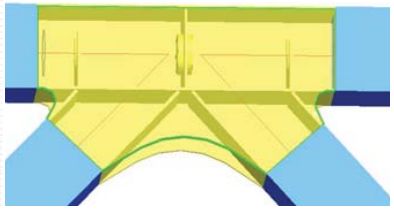
…….. or maybe this?



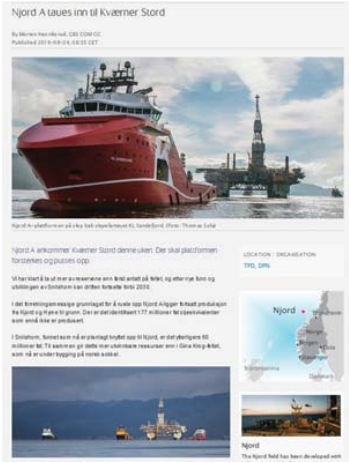
Simplicity is a quality!

- Clear load path – No eccentricities
- Change of capacity may be obtained without changing geometry
- Essential joints designed in FEED (lifting-nodes, support-nodes etc).
- Design was performed by use of pencil, HP-calculator and experience.
- Unchanged all through detailed engineering and construction

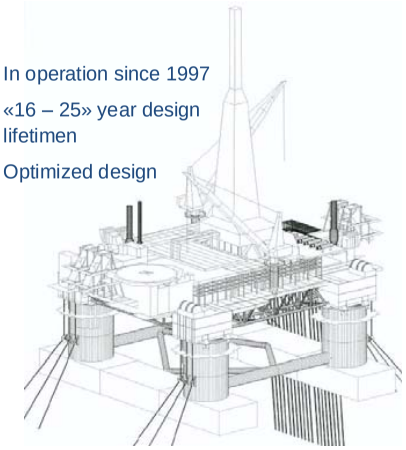




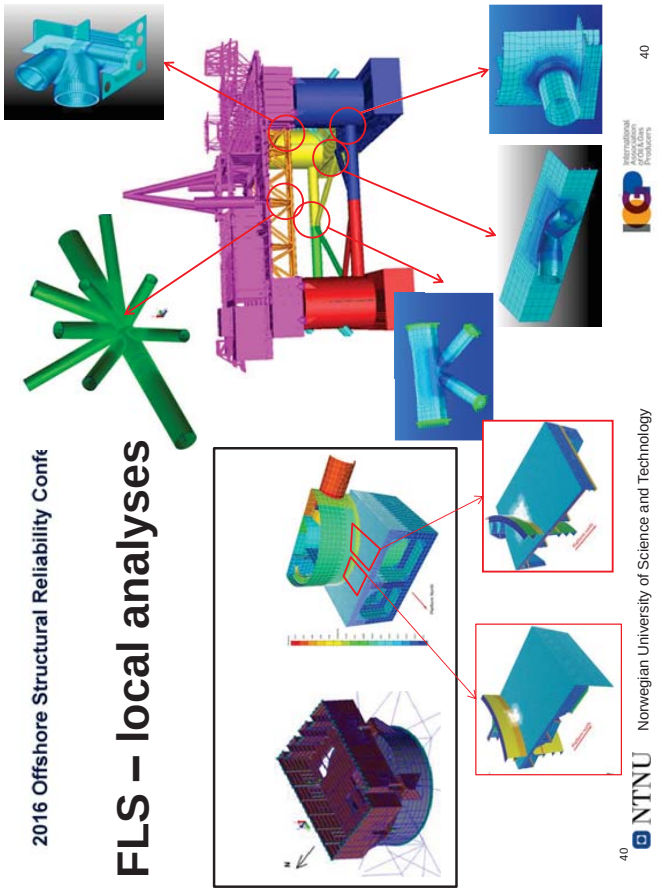
Njord A to shore

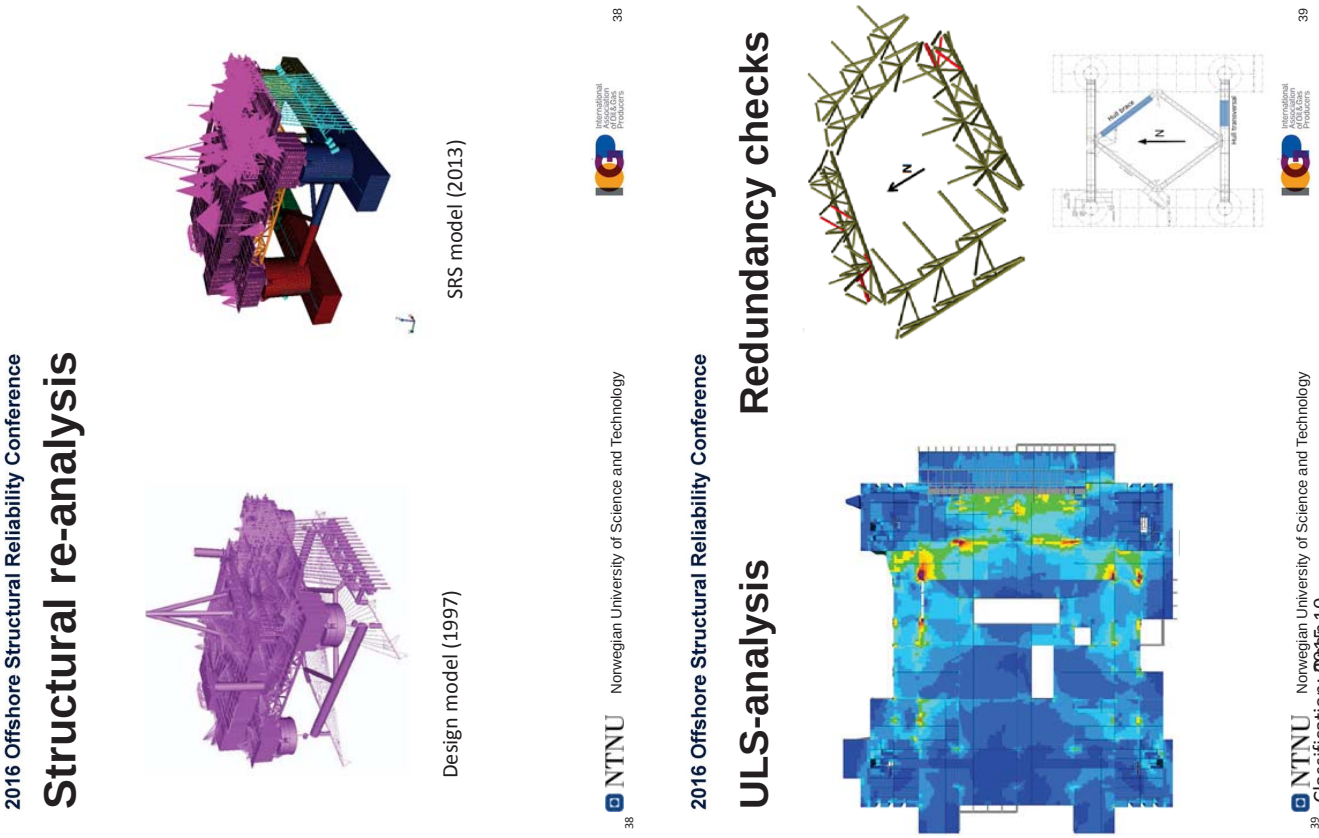


- In operation since 1997
- «16 – 25» year design lifetimen
- Optimized design



Deformations in deck structure





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Gudrun Helideck

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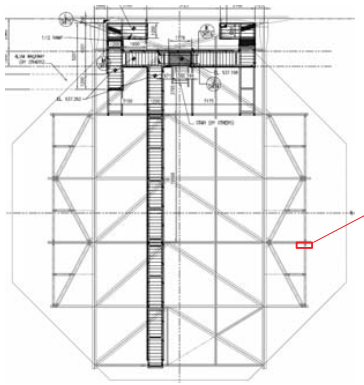
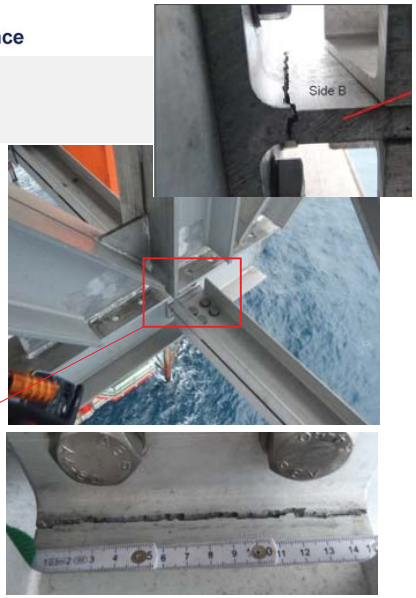
T-stub fracture due to VIV





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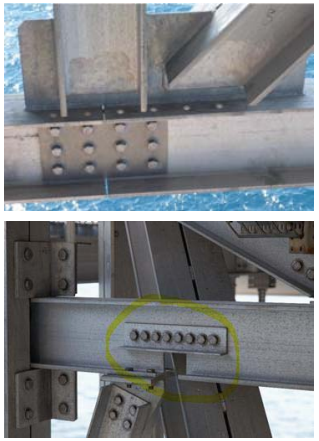
Poor detailed design – «Improvisation»



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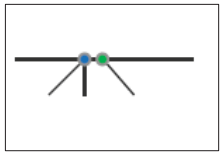
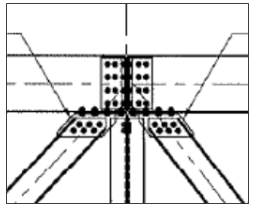
Challenging to document integrity

- The extensive use of bolted connections combined with poor detail design and discontinuities is demanding to analyse.
- The force transfer in the structure is not modelled correctly in way of bolted joints.
- The introduction of hinges at the joints will require a gap between crossing flanges.





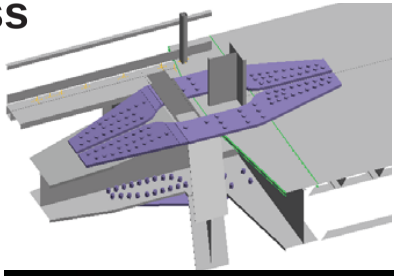
Reason for «loss of performance»:
Not necessarily lack of capacity but lack of ability to document

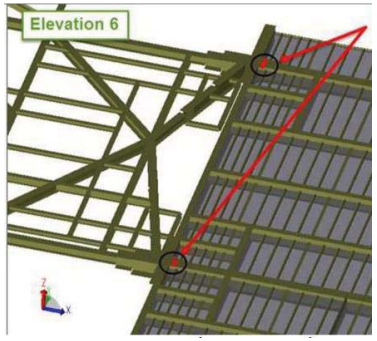
Re-opening - April 2016

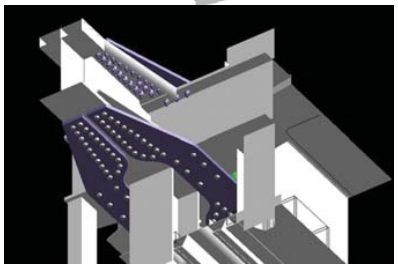
What characterizes a reliable structure?

Increasing robustness

- Deck fixed to LQ structure by aluminum weld
  - No overmatch: Weld weakest link
  - Brittle failure mode
- Redundancy introduced by bolted fish plate







What characterizes a reliable structure?

- Robustness
- Safety in design
- Simplicity
- Documentability

2.3 Keynote 2 by Philip Smedley

NTNU Norwegian University of Science and Technology

International Association of Oil & Gas Producers

Should Global Standards Specify Reliability Values for Design and Assessment?

Philip Smedley, Advisor Structures, BP
Here representing ISO Offshore Structures Committee

email: philip.smedley@uk.bp.com

The 3rd Offshore Structural Reliability Conference
OSRC2016
14-16 September, Stavanger, Norway

Introduction

- Very simple question:
  - Should Global Standards Specify Reliability Values for Design and Assessment?
- Of all topics discussed by offshore structural engineers, in my opinion it is the most divisive and has broadest range of views.
- In this lecture I will present the current consensus view within ISO, and look at some of the challenges around this topic.

Start with a Question…..

- I will offer you 1 of 3 identical fields for $1b, A, B & C
  - You can only have one but you can change your mind later…
  - Only 1 has oil and lots of it!
  - What's the chance of you selecting the winner?
- Tell me your answer and from the other 2 options I will eliminate one losing option and offer you the same choice between 2 remaining (one yours : one mine).
  - What is the chance of getting the winner if you stick with your choice?
  - Do you want to change your mind?

ISO Offshore Structures Standards

- ISO Standards committee for all types of offshore structures used in oil and gas industry are covered within ISO TC 67 / SC 7 "Offshore Structures".
- Strategy level committee comprises representatives from 22 Member Countries.
- Oversees 10 active Work Groups performing technical work for 20 Standards
- ISO 19900 series comprise 17 of these Standards (excluding Offshore Containers)

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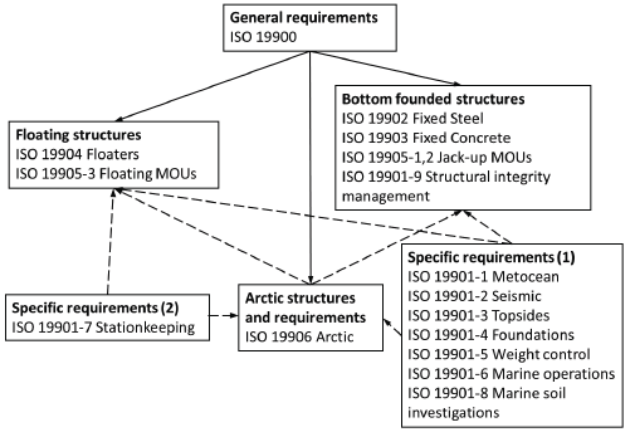
ISO TC 67 / SC 7 "Offshore Structures" ISO 19900 Suite of Standards



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Use of reliability tools in 19900 series

19900 General Reqs	ISO 2394 General Principles on Reliability of Structures Three levels of consequence (exposure levels) L1 – L3 Allows for different level of QMS No reduction in safety factors in service for L1 structures
19902 Steel structures	Calibration of component design factors to API 2A WSD plus North Sea environment action factor.
19903 Concrete structures	Design builds on national standards, esp NS 3473
19904 Floating structures	Design builds on Class Society Rules
19905 MOU structures	Calibration of component design factors to SNAME 5-5A with benchmarking to good practice.
19906 Arctic structures	Calibration based on ISO 19902, 19903, 19904 components plus additional action factors to give overall system reliability based on expert specified bounds and targets.
19901-2 Seismic	Specific probability of 'structural failure' stated from which return period seismic events derived.
19901-7 Stationkeeping	Checked to provide sufficient nominal safety

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ISO TC 67/SC 7 Current Position

- Formal Resolution created in 2008 and reaffirmed in 2013: "*SC7 .. confirms that the structure-specific ISOs produced by SC7 <u>cannot require or recommend specific target reliability levels</u>. General discussion around probability of component and system failure can be included in informative annexes with appropriate caveats around the assumptions and limitations of such information.*"

- Note that calibration of safety factors in ISO 19900-series using reliability methods is very much allowed and endorsed.

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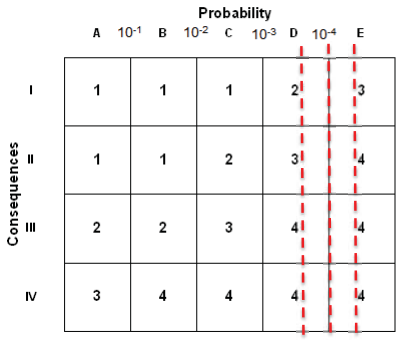
Corporate Risk Assessment



SHE	Envi	Public	Monetary
Fatalities	Widespread Environmental	Significant Public Disruption	Corporate financial impact
Serious Injury	Localized Environmental	Short duration significant media	Business unit impact
Medical	Minor	Local media	Facility economic impact
Minor	Inconsequential	Minor	Minor

From Ward Turner OSRC #1 Keynote, 2012

n.b. Some companies and industry sectors truncate low probabilities

Thus, help increasingly sought to determining probability of failure (see OSRC#1 report and do your own case-specific assessment)

Do we understand the challenges?

- Andrew Palmer[1] when discussing probabilities well into the tail of distributions, comments:
  - *"Only the most conscientious and unusual user would embark on the SRA approach, and if he were to do so he would soon encounter difficulties that could not be surmounted* ***in any honest way****."*
- 19901-7 Stationkeeping
  - *The actions on the floating structure … may be ignored .. provided that … the joint probability of occurrence of the adverse [ULS] environmental event combined with the failure to disconnect is less than 10^{-4} p.a.*
  - *OK : Given ULS RP = 100 years all I need do is trial 100 disconnections in some way and if all are OK that's good enough to meet Standard.*
  - Wrong on many levels – particularly independent vs conditional probability.

[1] "What do failure probabilities mean?, A Palmer, Keppel Professor, NUS, Pipelines Int., March 2013.

Specification of Reliability Level(s)

- Approx 10,000 offshore platforms of all types.
- Approx 1,500 could be considered manned.
- Based on very limited funding where to draw the line?
  - Differentiate by consequence?
  - Definition of failure?
  - Target or Limit?
  - Base on all hazards or a subset?
  - Benchmark to historic platforms?
  - Same all Regions?
  - Based on what model?

1,E-08 1,E-07 1,E-06 1,E-05 1,E-04 1,E-03 1,E-02 1,E-01 1,E+00

Specification of Reliability Level(s)

- Approx 10,000 offshore platforms of all types.
- Approx 1,500 could be considered manned.
- Based on very limited funding where to draw the line?
  - Differentiate by consequence?
    - Lives at risk – factor in per life, or bands of fatalities, or any life?
      - Typically 5 to 200 lives at risk (higher if multi-asset hazard)
    - Potential for environmental damage: oil vs gas
      - Reliance on shut down valve, non-free flow of hydrocarbon, etc
    - Pure financial loss – should Standards specify limits?
      - Costs could be beyond single organisation.
      - Reputation cost can be unpredictable.
  - ISO currently differentiates L1, L2, L3 (somewhat piecemeal)

1,E-08 1,E-07 1,E-06 1,E-05 1,E-04 1,E-03 1,E-02 1,E-01 1,E+00

Specification of Reliability Level(s)

- Approx 10,000 offshore platforms of all types.
- Approx 1,500 could be considered manned.
- Based on very limited funding where to draw the line?
  - Definition of failure (structural, lives, injury, spill bbls, cost$)?
    - Tend to discuss in terms of Probability of Structural Failure
    - Is it more appropriate to consider per annum or operational life?
    - Failure when exceeding a limit state?
    - Bottom founded structures: first component failure (exceed allowable, yield, break) and system collapse (write-off, partial or complete loss?)
    - Floaters: permanent list; hull penetration, 2 or more compartments; penetration of tank; loss of 1, 2, all moorings, damage/break to riser?
    - Easier for gross failure : Case specific for partial failure modes.

1,E-08 1,E-07 1,E-06 1,E-05 1,E-04 1,E-03 1,E-02 1,E-01 1,E+00

Specification of Reliability Level(s)

- Approx 10,000 offshore platforms of all types.
- Approx 1,500 could be considered manned.
- Based on very limited funding where to draw the line?
  - Target or Limit?
    - Targets are by definition vague (ISO 2394)
    - Limits – not to exceed, are very restrictive
    - ISO 19906 calibration example
      - target specified for objective function
      - limit set for range of cases considered (more cases = more likelihood one will exceed notional limit, i.e. Limit has some Prob of exceedance!)
      - One order of magnitude between target and limit.
  - Both: limit all platforms, target that reflects current good practice

1,E-08 1,E-07 1,E-06 1,E-05 1,E-04 1,E-03 1,E-02 1,E-01 1,E+00

Specification of Reliability Level(s)

- Approx 10,000 offshore platforms of all types.
- Approx 1,500 could be considered manned.
- Based on very limited funding where to draw the line?
  - Base on all hazards or a subset (natural, deliberate, errors)?
  - ISO 2394:
    - Class 1 includes storm, seismic, collision, fire/blast, tsunami, …
    - Class 2 includes vandalism, public order, (terrorism?)
    - Class 3 includes design/assessment error, material error, construction error, user error, lack of maintenance error, errors in communication.

1,E-08 1,E-07 1,E-06 1,E-05 1,E-04 1,E-03 1,E-02 1,E-01 1,E+00

Specification of Reliability Level(s)

- Approx 10,000 offshore platforms of all types.
- Approx 1,500 could be considered manned.
- Based on very limited funding where to draw the line?
  - Same all Regions? (use cost-benefit approach?)
    - ISO 2394 recommends using a life quality index (LQI) approach to help specify life-safety optimal points based on society's ability to pay for further safety measures.
      - Rich country sets high reliability for a hospital which poor country cannot afford to meet so hospital goes unbuilt and society loses out.
      - Is an offshore platform really different – where does the money come from to build the hospital?
    - Could help National/Regional Regulators in their deliberations?

1,E-08 1,E-07 1,E-06 1,E-05 1,E-04 1,E-03 1,E-02 1,E-01 1,E+00

Benchmark to Platform Failure Rates

Year	Where	Name	Type	Lives	Loss	Cause
1980	**Norway**	**Alexander Kielland**	**Semi (floatel)**	**123**	**Capsize**	**Fatigue**
1989	Thailand	Seacrest	Drillship	91	Capsize	Typhoon
1982	**Canada**	**Ocean Ranger**	**Semi (drilling)**	**84**	**Capsize**	**Storm**
1983	China	Glomar Java Sea	Drillship	81	Capsize	Storm
1979	China	Bohai 2	Jack-up (Tow)	72	Capsize	Storm
2011	Russia	Kolshaya	Jack-up (Tow)	53	Capsize	Storm
2005	**India**	**Mumbai High North**	**Jacket**	**22**	**Collision PSV**	**Blast from Riser**
2007	**GoM**	**Usumacinta**	**Jack-up**	**22**	**Collision Jacket**	**Storm Fire**

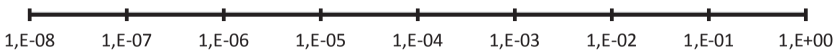
Data only for >20 fatalities

very vague vs latest practice

1,E-08 1,E-07 1,E-06 1,E-05 1,E-04 1,E-03 1,E-02 1,E-01 1,E+00

Specification of Reliability Level(s)

- Conclusion:
  - It is feasible to specify one/multiple reliability 'target(s)', but
  - Broad range of assumptions can form this 'target'
  - Too often there is a lack of detail over the basis for meeting such a 'target'
  - Have we (assessors/assurers) sufficiently competent in complex probability engineering?
  - Do we sufficiently recognise wide degree of uncertainty in derived reliability values?

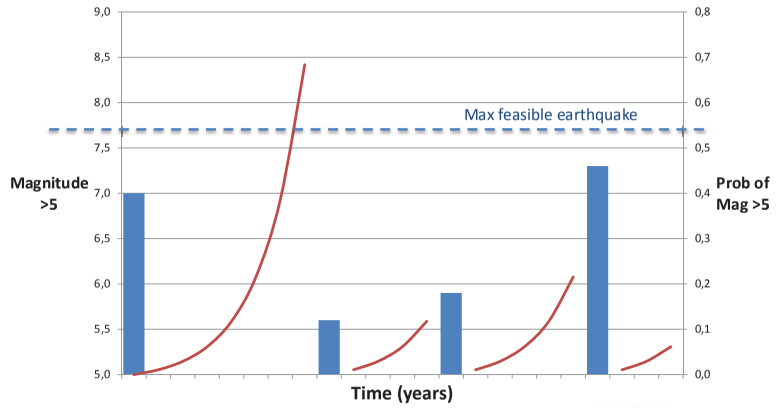


Low Probability Events - Seismic

- Concerns over ISO quoted Pf = 2500 years (4 x 10^{-4} p.a.)
- Return period ALE 3000 – 5000 yrs - fully utilised platform
- Unpredictable but not independent or infinite magnitude
  - *Physical build-up of plate movement that breaks out when frictional resistance overcome;*
  - *A maximum seismic event can exist;*
  - *If a big event happened 'recently' the high magnitude event less likely to occur;*
  - *2500 yrs chosen as reflects written records or major events;*
  - But, multiple facilities can be affected – even higher consequence
  - But also, but probably not a disaster limited to offshore. Will there be a emergency response to offshore?; availability of hospitals?; availability of office support?; effect on offshore platforms may not even make that day's news.

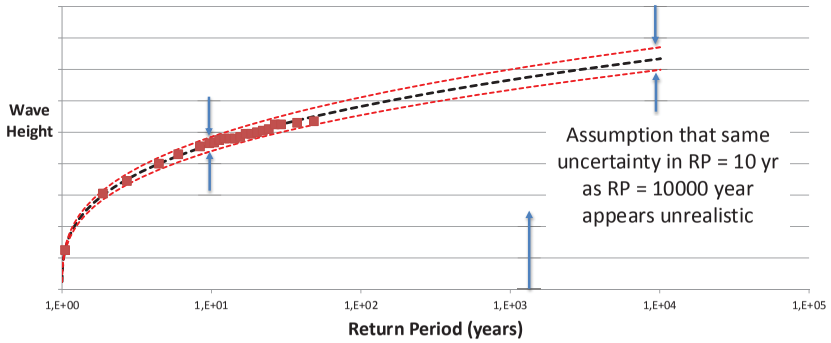
Low Probability Events - Seismic

- Unpredictable but not infinite magnitude nor independent



Low Probability Events - Metocean

- Major debate when design RP extended from 50 to 100 yrs
- Applying 1000 yr to 10000 yr extrapolated values now becoming the norm!



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Low Probability Events - Metocean

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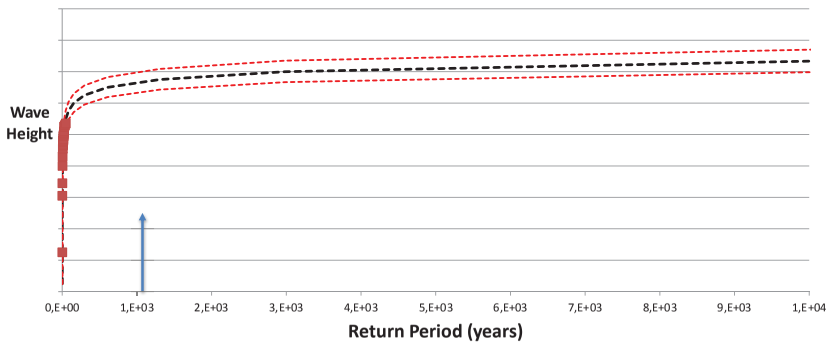

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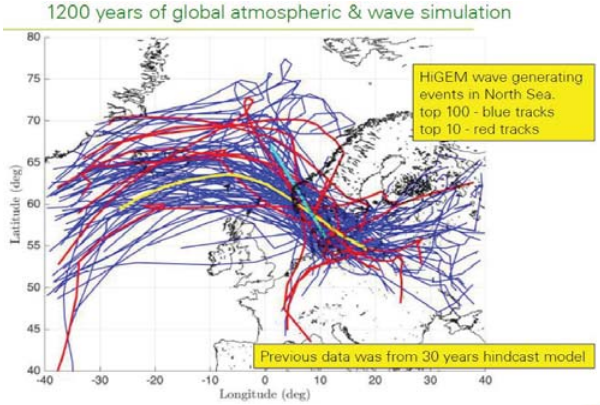
Low Probability Events - Metocean

- What would a RP = 10000 years (10^{-4} p.a.) storm look like?
- How would industry react to such a developing hazard?



From
Oliver Jones and
Ramsay Fraser, BP

2016 Offshore Structural Reliability Conference

Low Probability Events - Collision

- My personal biggest fear = Large vessel going full speed into manned platform – probability?

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Closure

- Reliability assessment is a valuable tool to help calibrate to existing specifications.
- Reliability assessment is a useful tool for organisations to estimate magnitude of risk and rank mitigation measures.
- Significant concerns remain around specification of 'targets' and assurance against 'target' values:
  - Standards can provide more guidance
  - Common (default) basis for assessment may be desirable
- For very low likelihood events, emphasis must remain on ability to withstand hazards: robustness, effective QA/QC, proactive inspection & maintenance (SIM system), downmanning, sharing lessons, plans for 'what-if' cases..

2.4 Keynote 3 by Dave Wisch

NTNU Norwegian University of Science and Technology

International Association of Oil & Gas Producers

API 2GEN – Overarching Document for Developers of the API Series 2 Standards

David Wisch
Chevron Energy Technology Company
(Presented by Andrea Mangiavacchi, EXPERIA LLC)

The 3rd Offshore Structural Reliability Conference
OSRC2016
14-16 September, Stavanger, Norway

API RP 2GEN (GENeral Principles)

- Publication of ISO 19900
- Historical review - 70 years of offshore structures
- API SC2 design practices developed "as needed"
- 20+ years of ongoing discussions within US community (with international participation):
  - Why do we do things the way we do?
  - What are the key principles?
  - Where do the "design recipes" come from?
  - Are there "hidden assumptions"? Can we document?
- **=> REVERSE ENGINEERING EXISTING STANDARDS**
- **Provide basis for new standards**

2GEN Objectives

- **Focus on API SC2 Document Portfolio**
- **Create a high-level framework for "lower" documents**
  - Create consistency among all documents residing below 2GEN
  - Clarify Categorization and Design Criteria of Facilities
    - Represented as "Design Scenarios", "Design Conditions", etc.
      - Manned / Unmanned
      - Operational / Shut In
      - Restricted / Unrestricted
      - Economic
  - Ensure consistency between floating and fixed structures
    - Point to same type of checks (i.e. life safety, robustness, etc.)
    - Logic needs to work on both
    - Performance Levels / Limit States
- **Address Life Cycle**
  - New Design, Pre-Service, Assessment, Beyond "Design Life"

Process

- Form Dedicated Small Team (ca 2008-9)
- Review/Audit Existing API SC2 Documents and TG/RG
  - Review design recipes
  - Identify Defined Loading Conditions and Acceptance Criteria
  - Associate "Restrictions"/"Conditions" for Each Set
  - Record Stated/Written/Explicit Design Cases
  - Note "Generally Accepted Industry Practice" not in Documents
- Pattern Findings
- Establish Framework (Generalize ISO 19900?)
- Test
- Wide Review and Comment
- Publish New RP

Perceived Current Challenges - 1

- Categorization of facilities
  - Life safety
  - Hydrocarbon Operations/Containment
  - Economic Consequences
- Determination of Exposure Levels
- Structural Performance Levels
- Acceptance Criteria
- Review of present design criteria and loading conditions
  - Return periods vs. performance
  - Acceptance/performance vs. Design Scenario (restrictions)
  - Etc.
- Difference between assessment and new build

Perceived Current Challenges - 2

- **Economic Consequences highly variable**
  - **Extremely High (hub platform for deep waters or LNG)**
  - **High**
  - **Medium**
  - **Low (single well braced monopod)**
- **Managing Consequences (not well addressed)**
  - **Compliance with Standards**
  - **Manning/Evacuation**
  - **Operational Restrictions (e.g. no drilling in some cond.)**
  - **Changes of configuration (Disconnectable FPSOs)**
  - **Pre-service Conditions**
- **Fixed steel structures in GoM – easy (…?)**
  - **Unmanned for ULS**
  - **Low slope of hazard curve for winter storms/sudden hurricanes**
- **Floating structures – additional, more diverse failure modes**
  - **Excessive offset**
  - **Loss of Stability, capsizing**
  - **Etc.**

Perceived Current Challenges - 3

- Need for Evolution – Change way of thinking
- Inertia – Resistance to change
- Limited Experience/Perspective
- Component Methodology
  - System Effect – ULS to ALS
  - Ill Defined ALS
- Typical Limit States
  - SLS, ULS, ALS, FLS ?
- Risk Consistency
  - Across documents
  - Difficult alignment - Company Internal Risk Matrices
- Assessment, Extending Service, New Design
- Other

Perceived Current Challenges - 4

- Alphabet Soup & Definitions Complex and Confusing
  - L1 for jacket not similar consequence as for TLP or Semi
  - L1 applied to Station Keeping – Permanent vs. Mobile (MODU)
  - L1 as currently defined
    - **may or may not include consideration of Life Safety**
    - Not manned for one hazard curve
    - Manned for second curve
    - Widely different economics
- Risk valuation not consistent with ISO 2394:2015
  - 2394 has only two offshore references
  - Does not address breadth of industry needs

Audit Observations 1

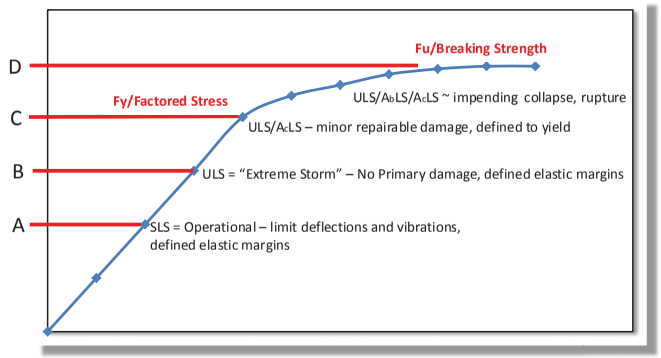
- ULS conditions exist in all form documents (RP 2A, RP 2FPS, RP 2SK and RP 2T)
  - consistent in performance expectations at defined load recurrence with resistance at the "B" level (see later slide)
- Operational conditions, SLS, clear performance criteria, load recurrence is variable
- The ULS condition for Station Keeping has inherent variability (all lines intact and a one line damaged/out of service condition)
- AcLS situations have little explicit load or performance criteria.
- AbLS conditions - increasing number and return periods with limited general criteria for acceptability:
  - 2A and 2T deck clearance requirements
  - 2T has AbLS condition for loss of tendon tension but no other checks.
- **Reliability has been achieved via the elastic component design approach calibrated to performance expectations**

Audit Observations 2

- Multiple Design Objectives – many "hidden" in design practice
- Consequence Based Criteria (CBC) – "simple" in 1992, broad moving forward
- Need to Partition Design Conditions
  - Life Safety, Environmental, Economics/Headline
  - Not Lumping Into Single H, M, L
- Explicit consideration
  - Load Recurrence
  - Acceptance conditions
- Express as Return Period and Acceptance
- High Level Consistency
- Implementation in Specific Documents
- Address Design and Assessment
- Simpler Assessment In the Future

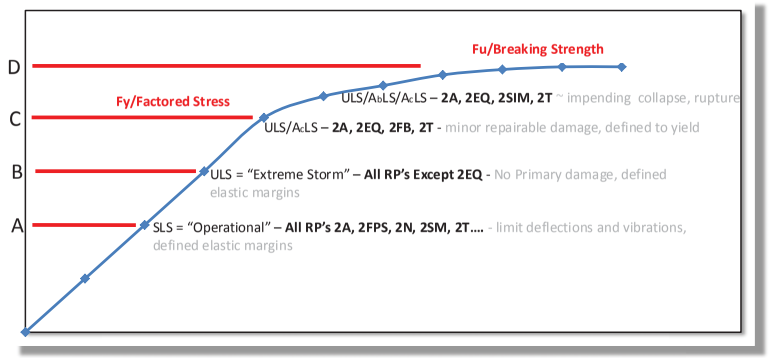
2GEN Structural Performance Level

This is what is currently done

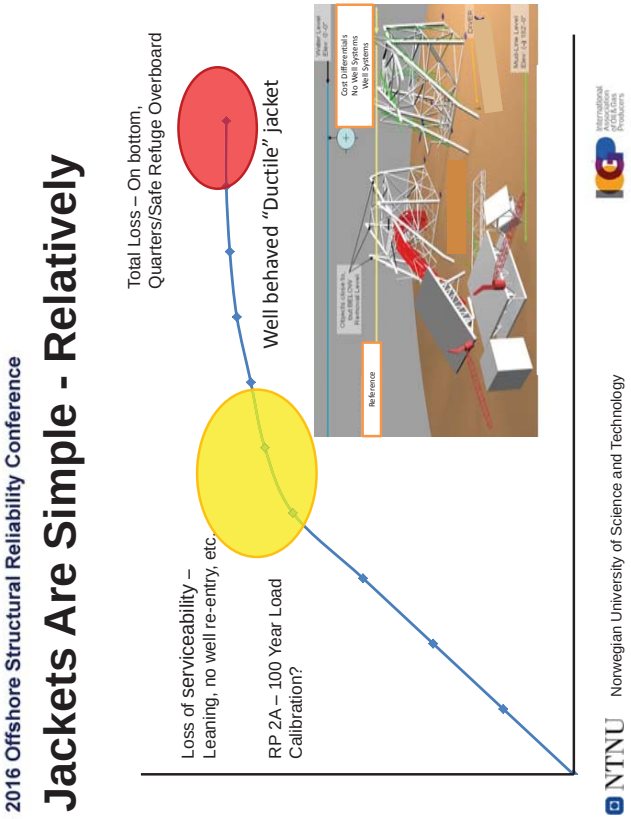


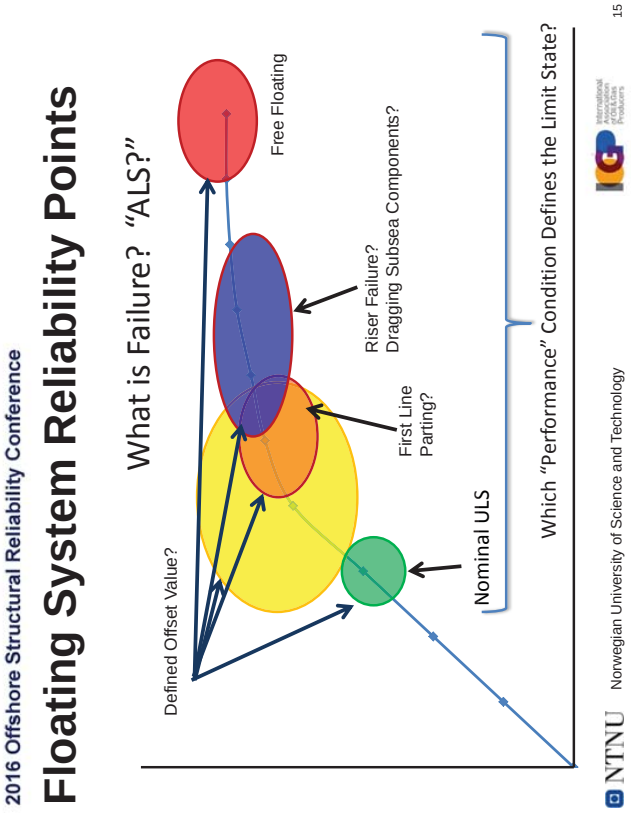
ACCEPTANCE POINTS – some implicit, some hidden in equations and partial factors, some by explicit value, some with different names

Structural Performance Level's by RP



ACCEPTANCE POINTS by API SC2 Documents


2016 Offshore Structural Reliability Conference
Jackets Are Simple - Relatively
Total Loss – On bottom, Quarters/Safe Refuge Overboard
Well behaved "Ductile" jacket
Loss of serviceability – Leaning, no well re-entry, etc.
RP 2A – 100 Year Load Calibration?
Reference
Cost Differentials
No Well Systems
Well Systems
NTNU
Norwegian University of Science and Technology
15


2016 Offshore Structural Reliability Conference
Floating System Reliability Points
What is Failure? "ALS?"
Defined Offset Value?
Free Floating
Riser Failure?
Dragging Subsea Components?
First Line
Parting?
Nominal ULS
Which "Performance" Condition Defines the Limit State?
NTNU
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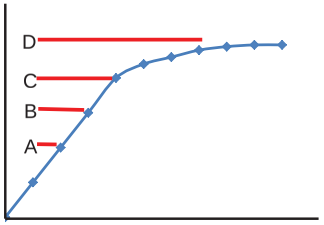
15

2016 Offshore Structural Reliability Conference
Jacket Reliability Points
Highly Ductile
Total Loss – On bottom, Quarters/Safe Refuge Overboard
Loss of serviceability – Leaning, no well re-entry, etc.
RP 2A – 100 Year Load - ULS
Brittle – K or Diagonal Brace
Reference
Cost Differentials
No Well Systems
Well Systems
NTNU
Norwegian University of Science and Technology
14

2016 Offshore Structural Reliability Conference
Present Gulf of Mexico Practice
Structural Performance Level
Limit State
Recommended Practice
Return Period
Restricted/ Unrestricted
A
SLS
All RP's:
2A, 2FPS, 2N, 2SM, 2T....
1-10 yrs
UR
B
ULS
All Except 2EQ
100 yrs
UR or R
C
ULS/ALS
2A, 2EQ, 2FB, 2T
200-1000 yrs
UR or R
D
ALS
2A, 2EQ, 2SIM, 2T
>1000 yrs
UR or R
Note: RP2A 22nd uses 5 Design Conditions
•LC1 ~ A
•LC2 ~ A
•LC3 ~ B
•LC4 ~ B
•Deck Elevation ~ C
A
B
C
D
NTNU
Norwegian University of Science and Technology
16

Some Thoughts

- ULS – Fitness and Performance – SLS & ULS
- Design for Purpose
  - Return Load Period
  - Performance for Load Condition
- Design for Condition Assessment
  - Can be factored from design
  - Can be "checked"
- Reliability
  - Performance must be explicit
  - Sensitive to assumptions
    - Extrapolation, condition, hazard curve, linearity, etc.
- Reliance Upon Jacket Knowledge
  - False security
  - Applicable



Practice – Loading/Actions & Acceptance

Note (1) - Correlate to the frequency of the incident and the deired performance level
Note (2) - Correlate the severity and the event and the desired performance level (See API 2EQ & ISO 19901-2)
Note (3) - Correlate to the duration of the tow & desired performance level prescribed (see API 2MOP and ISO 19901-6)
Note (4) - Correlate to the sensitivity of the operation & desired performance level prescribed (see API 2MOP and ISO 19901-6)
Note (5) - Correlate to the sensitivity of the operation, exposure duration, desired performance level and sensitivity to single design event
Note (6) - Expected metocean conditions during life with appropriate factors with "Crack" referenced at C level with stated margins for acceptance
Note (7) - Correlate the return period of the event and the desired performance level (C) to achieve "fit for purpose" repairs
Note (8) - Factored as appropriate as values are not return period or recurrence based

Practice – Loading/Actions & Acceptance

Example

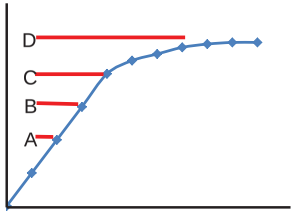
Scenario/Condition	Exposure	Return Period	Performance level
Operating / SLS - Storm	Metocean Conditions	1 - 10 yr	A
Extreme / ULS - Storm		100 yr	B
AbLS - Abnormal - Storm		1,000-10,000 yr	D
Accidental	Blast	1,000-10,000 yr	C & D
	Fire	1,000-10,000 yr	C&D
	Dropped Object	As Specified (1)	A, B, C & D
	Collision	As Specified (1)	C & D
Extreme / ULS Earthquake (ELE)	Earthquake	As Specified (2)	C
AbLS - Abnormal - Earthquake		As Specified (2)	D
Pre-Service	Fabrication	1 - 10 yr	A & B
	Loadout	1 - 10 yr	A & B
	Transportation	1 - 100 yr (3)	B & C
	Installation	<1 - 100 yr (4)	B & C
	Fatigue	As Specified (5)	C (7)
Fatigue - FLS	Mean MetOcean Scatter	Facility Life (6)	C (7)
"Damage"	Metocean Conditions	1 - 100 yr (7)	A, B & C
Dead and Live Loads/Actions	As defined	Mean Static Loads/Actions (8)	A, B, C & D

From

- Alphabet Soup
  - L1, L2, L3, A1, A2, A3, S1, S2, S3, etc.
- Variance of design conditions
- Subsystem approach
  - FPS
  - RD
  - SK, SM, etc
- Independent documents vs. document suite
  - MET, GEO, EQ, TOP, MOP, etc.
- Implicit Conditions

To

- Systems Linkage
- Explicit Performance Expectations
- Recurrence Linked to Performance
- Explicit Design Conditions/Scenarios
- Explicit Decisions
- Conscious Risk Management
- Design and Assessment Correlation
- Associate Acceptance/Recurrence with Performance
- Establish "Design Conditions/Scenarios"
  - Use "Restrictions" to align



Next Steps

- Receive and Address Comments
- Move to 19900:2015
- Rearrange/Modify/Eliminate
  - Restricted/Unrestricted
  - Event Categorization
    - What "Limit State" is a mooring "one line out of service" condition?
      - Damage Case?
      - Accidental Case?
      - What Performance Criteria? A? B? C? D?
  - Improve D Performance Level Explanation
  - Etc.
- Publish 2017

Summary – Value of New Framework

- Move away from Alphabet Categorization, which Addresses Only "Failure"
- Readily Adapts to Facilities with Multiple Hazard Curves
- Move to Functional Approach
  - Define Performance and Likelihood of Exceedance
  - Explicitly Address Range of Design Conditions
  - Lower Documents Define Details
- Permits Improved Balance Across Breadth
- Deviates from Jacket Centric Categorization
- Blends with Company Risk Management Approaches
- Improves Transparency
- Explicit Decisions

Key Challenges

- **Adjustment in Thinking**
- **Exporting/Extrapolating concepts to international arena**
- **International Sanctions and API-ISO divergence NOT HELPFUL!**

END

Thank you for your attention!

Chapter 3

Session 1: Metocean Conditions

3.1 Presentation & article by Kjersti Bruserud & Sverre Haver

NTNU Norwegian University of Science and Technology

International Association of Oil & Gas Producers

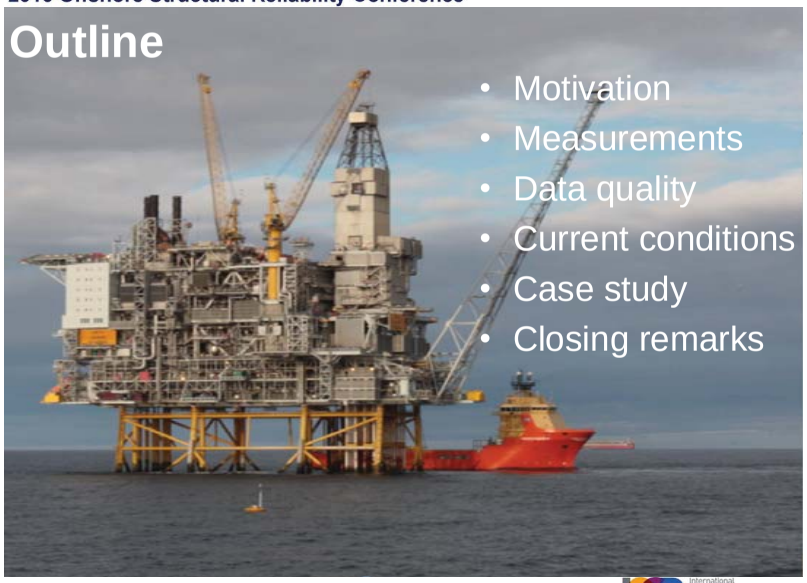
Waves and Associated Current

Experiences from a Five Year Measurement Campaign in the northern North Sea

Kjersti Bruserud, Statoil/NTNU
Sverre Haver, NTNU/UIS

The 3rd Offshore Structural Reliability Conference
OSRC2016
14-16 September, Stavanger, Norway

Outline



- Motivation
- Measurements
- Data quality
- Current conditions
- Case study
- Closing remarks

Motivation

NORSOK STANDARD N-003 (Ed.2, 2007)

6.7 Combination of environmental actions (Table 4)

Limit state	Wind	Waves	Current	Ice	Snow	Earthquake	Sea level[a]
Ultimate Limit State	10^{-2}	10^{-2}	10^{-1}	-	-	-	10^{-2}
	10^{-1}	10^{-1}	10^{-2}	-	-	-	10^{-2}
	10^{-1}	10^{-1}	10^{-1}	10^{-2}	-	-	m
	-	-	-	-	10^{-2}	-	m
	-	-	-	-	-	10^{-2}	m
Accidental Limit State	10^{-4}	10^{-2}	10^{-1}	-	-	-	m*
	10^{-2}	10^{-4}	10^{-1}	-	-	-	m*
	10^{-1}	10^{-1}	10^{-4}	-	-	-	m*
	-	-	-	10^{-4}	-	-	m
	-	-	-	-	-	10^{-4}	m

- Conservative?
- If yes, degree of conservatism?

Motivation

NORSOK STANDARD N-003

Actions and action effects

NORSOK STANDARD N-003 (Ed.3, 2016)

6.2.3 Modelling simultaneous occurence of metocean data

"Simultaneous wind, waves and currents should normally be taken in accordance with Table 9..."

"If current shall be included in joint modelling, simultaneous data must be used. The data must be site specific or documented to be representative for the area in question. Furthermore, the dataset must have sufficient duration in order to ensure that the long-term variability in storminess, seasonality and inter-annual variability are encountered for. **At least 5 years of simultaneous data are recommended to evaluate any reduction** .... relative to Table 9"

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Measurements

Objective

Sufficient simultaneous wave and current data, including LARGE storms, to establish joint distributions of extreme waves and currents

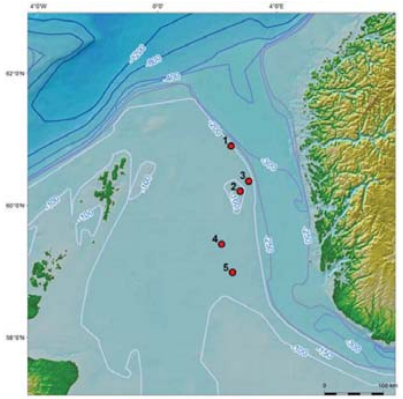
2016 Offshore Structural Reliability Conference

Measurements

Period
- 05.2011 – 10.2015, i.e. 4.5 years

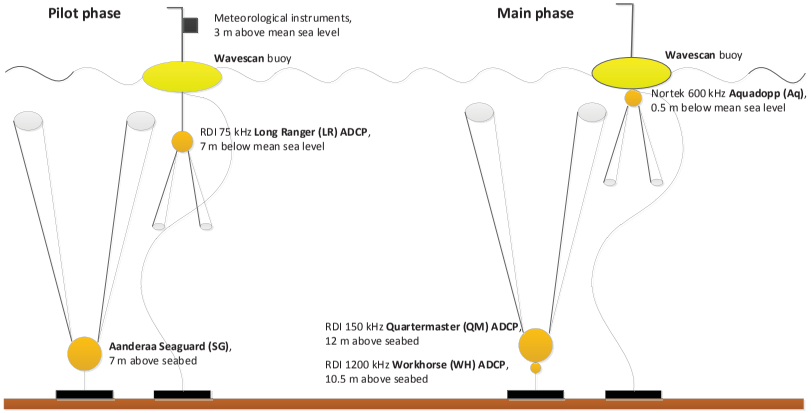
Data coverage
- 80 – 90 %

Measurement interval
- 10 minutes, currents
- 30 minutes, waves



2016 Offshore Structural Reliability Conference

Measurements



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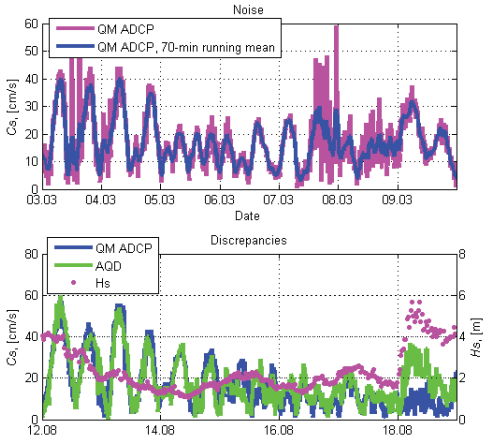
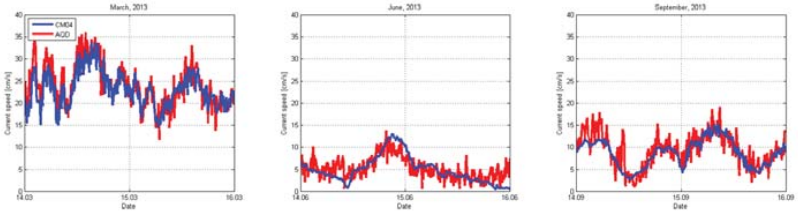
Data quality



Data quality

Current Verification Study (CurVeS), phase I, II & III

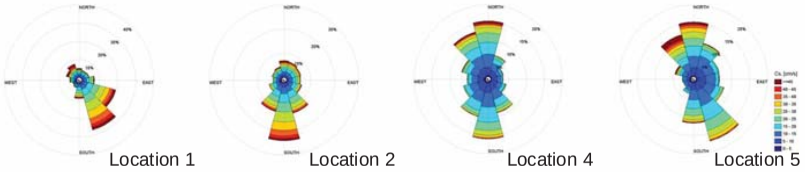
> "… differences in observed *Cs* are usually much larger than the specified accuracies of the instruments, suggesting that the accuracy achieved in the field are often much less than the user might expect."



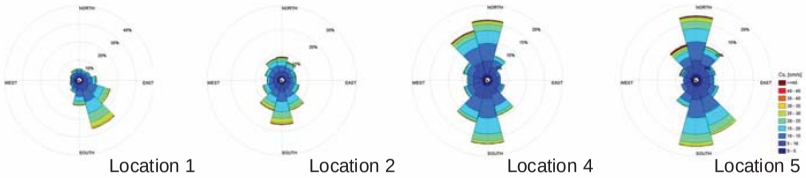
Current conditions

Seasonal variations

- Winter



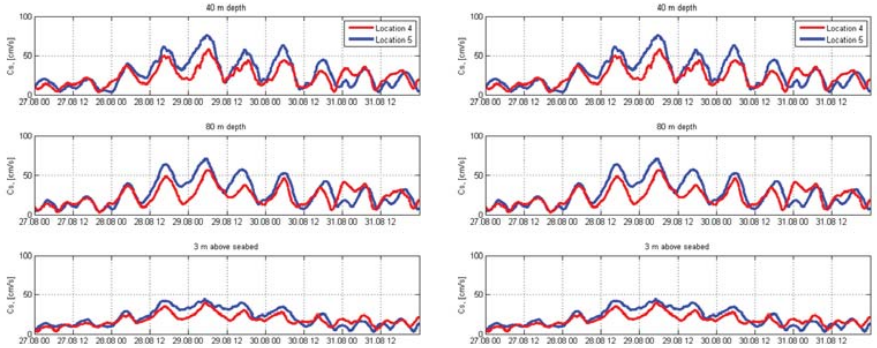
- Summer



Current conditions

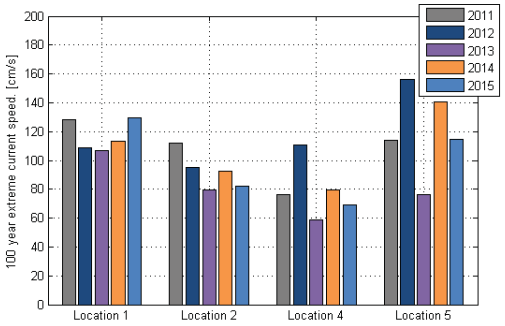
Seasonal variations

Location 4 and 5



Current conditions

Interannual variations



Case study I

Purpose

To investigate different combinations of wave and currents to estimate ULS load on a jacket

1. According to N-003
   - All-sea states
   - Peak-over-threshold
2. Directly from load time series
   - Most probable wave height
   - Monte Carlo simulated wave height

   Different averaging of current speed

→ Assessment of possible conservatism

Case study I

1. According to NORSOK N-003

Parameters		ULS load
All-sea states *H*	10-min *Cs*	1.000
	3-hrs mean *Cs*	0.953
	3-hrs max *Cs*	0.969
Peak-over-threshold *H*	10-min *Cs*	0.886
	3-hrs mean *Cs*	0.862
	3-hrs max *Cs*	0.877

2. Directly from load time series

Parameters		ULS load
Most probable *H*	10-min *Cs*	0.540
	3-hrs mean *Cs*	0.522
	3-hrs max *Cs*	0.559
Monte Carlo *H*	10-min *Cs*	0.623
	3-hrs mean *Cs*	0.599
	3-hrs max *Cs*	0.638

Observations

- All-sea states more conservative than peak-over-threshold
- Monte Carlo *H* more conservative than most probable *H*
- 3-hrs max *Cs* most conservative
- NORSOK N-003 more conservative than directly from time series

Please note: Illustrative purpose ONLY, NOT suitable for specific design.

Case study II

Location	Waves	Current	ULS load
1	All-sea states *H*	3-hrs max *Cs*	1.000
	Most probable *H*		0.696
2	All-sea states *H*	3-hrs max *Cs*	1.000
	Most probable *H*		0.603
4	All-sea states *H*	3-hrs max *Cs*	1.000
	Most probable *H*		0.564
5	All-sea states *H*	3-hrs max *Cs*	1.000
	Most probable *H*		0.540

Observation

- NORSOK N-003 more conservative than directly from time series at all locations

Closing remarks

Quality of current measurements

- Less accuracy than expected

Governing current conditions

- Wind-generated inertial oscillations

Extreme currents for design

- Large interannual variations

Potential in simultaneous wave and current data

- NORSOK N-003 seems to be conservative

Acknowledgements

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- Chief engineer Simen Moxnes secured Statoil's funding and this is gratefully acknowledged.
- Statoil is acknowledged for the permission to use the data and publish these results.

Waves and associated currents–experiences from 5 years metocean measurements in the northern North Sea

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Waves and Associated Current – Experiences from a Five Years' Measurement Campaign in the Northern North Sea

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ABSTRACT: In order to acquire sufficient simultaneous data to establish joint distributions of waves and currents for design, an extensive metocean measurement programme has been performed over a period of approximately five years at several locations in the northern North Sea. A brief description of the measurement programme is given. The measured current data have been found to be more inaccurate than the specified accuracy of the instruments. However, the measured current data still give a good over-all description of the current conditions. At the southernmost locations, wind-driven currents, i.e. inertial oscillations, are the governing current conditions and contribute to larger current speed during summer than in the spring and fall, both operational and extreme. At all locations, year-to-year variation in estimated extreme current speeds based on different individual years are larger than expected, indicating that current measurements for considerably more than one year is required for reliable estimates of extreme current conditions for design of offshore structures. Sensitivity studies of the ULS load on a jacket suggest the possible gain of accounting for the simultaneous occurrence of metocean parameters. The Norwegian design regulations seem to be conservative, at least regarding ULS. These results highlight the need for a better understanding of the current conditions in order to account for the uncertainties associated with these in design of offshore structures.

KEY WORDS: Current measurements; Wave measurements; Northern North Sea; Jackets.

1 INTRODUCTION

Characteristic meteorological and oceanic (metocean) loads and load effects are defined in terms of their annual probability of exceedance, q. The requirements for ultimate limit state (ULS) and accidental limit state (ALS) for characteristic metocean actions are $q \leq 10^{-2}$ and $q \leq 10^{-4}$, respectively. This requirement refers to the resulting metocean load, i.e. the characteristic metocean load obtained by accounting for the simultaneous occurrence of environmental parameters such as wind, waves and current. These parameters are not fully correlated and to utilize this for design, simultaneous data of both good quality and sufficient length are required.

In lack of sufficient simultaneous data, the Norwegian design regulations [1], recommend a combination of metocean parameters assumed to be conservative, but the degree of conservatism is not very well known. To utilize that the occurrence of extreme wind, waves and currents are not fully correlated in design of offshore structures, the latest edition of NORSOK STANDARD N-003 Edition 3 (N-003) [1] recommends at least three years of simultaneous wind, wave and current data.

In order to perform a more accurate analysis of marine structures, joint probability distributions of different environmental parameters have received an increasing interest during the last decade, facilitated by improved availability of reliable joint environmental data. Many recent studies have explored different bi- and multivariate statistical models for environmental parameters. For simplicity, bivariate statistical models are often presented rather than the multivariate generalizations, but these are easily extended beyond two dimensions to multivariate models.

Joint distributions of different environmental parameters based on a marginal distribution of the primary parameter and a conditional distribution for the associated parameters are frequently used and also adopted in design codes. Joint distributions of significant wave height and wave period, both zero up-crossing and peak period, are extensively studied and numerous approaches for Norwegian waters are available in the literature [2-7]. However, the joint environmental model proposed by Haver [8, 9] based on a marginal hybrid lognormal-Weibull distribution of significant wave height and a conditional lognormal distribution of spectral peak period is widely accepted and used. Later, this joint description of significant wave height and spectral peak period was extended to include wind speed, storm surge and current speed, all these parameters conditional on significant wave height and modelled with a normal distribution [10].

The semi-parametric conditional extremes model introduced by Heffernan and Tawn [11] has been strongly recommended for estimation of joint distributions of metocean parameters through a series of studies where this model has been adopted and applied in different ocean basins, including the northern North Sea [12-16]. Bivariate modelling of different combinations of environmental parameters have been performed based on a wide range of bivariate parametric probability distributions, see for instance [17-22]. The use of copula techniques has become increasingly popular and a number of studies has proposed bivariate models for different pairs of metocean parameters based on different copula

techniques in different worldwide ocean basins [23-27], including the North Atlantic [28].

There are few available studies of the joint probability of waves and currents in the surface of the water column, probably due to the lack of simultaneous measured wave and current data and the complicated, far from fully understood, wave-current interaction mechanisms. However, based on simultaneous wave and current measurements at Tromsøflaket, both Gordon, Dahl [29] and Heideman, Hagen [30] investigated the relationship of extreme waves and currents and established very simplified joint distributions to be used in design and load calculations for offshore structures. Wen and Banon [31] developed a probabilistic methodology that lead to joint probability distributions of hurricane induced winds, waves and currents at a generic site in the Gulf of Mexico. Prior-Jones and Beiboer [32] estimated joint design criteria for current speed and waves in the southern North Sea and highlighted the need to develop sound design practices for application of the joint environmental probability factors. Based on simultaneous metocean measurements in the Northern North Sea, joint probabilistic models has been proposed for waves and current [33] and wind and waves [34].

However, there still seems to be no general consensus with regard to the approach of estimating the joint probability distributions of environmental parameters and several different approaches are put forward. Jonathan and Ewans [35] gave a good theoretical overview of multivariate modelling of extreme ocean environments and guidelines for validity, but pointed out that "unfortunately there is as yet no unifying approach, and the literature is rather confusing". Ewans and Jonathan [15] concluded that specification of joint design criteria has often been somewhat ad hoc, based on experience and intuition and thus fairly arbitrary combinations of independently estimated extreme values. Vanem [28] demonstrated that there were large variabilities and thus large uncertainties in the estimated joint models due to different modelling choices, even for one the same data set, and concluded that multivariate modelling of metocean conditions remains a challenge, even in the bivariate case.

For the Norwegian waters, wind and waves data covering several decades are available, but currents are rarely measured for a period longer than one year. Following this, the limiting factor for a robust joint consideration of wind, waves and currents is the short duration of available current data. To secure simultaneous wave and current data for estimation of joint distributions for design of offshore structures, a metocean measurement programme at five locations in the northern North Sea was initiated early 2011 and completed late 2015.

This paper provides a brief description of the metocean measurement programme and highlights the challenges related to the quality of measured current data. Next, the variations in current conditions in this part of the northern North Sea are described. The possible conservatism in the Norwegian design regulations and thus the potential in utilizing simultaneous waves and currents is illustrated for a selected platform case based on the measured wave and current data from the northern North Sea. At last, a summary is made.

2 MEASUREMENTS

A metocean measurement programme of simultaneous waves and current profiles at five locations in the northern North Sea was initiated early 2011, see Figure 1.

First, a pilot phase was performed at Location 1 from January to May 2011, before the main phase with measurements at all five locations started in May 2011. At Location 3, the measurements were ended late 2013 and will not be considered in this paper, but at the other locations the measurement were completed in October 2015, i.e. a total duration of about 4.5 years. An overview of the water depths and data returns are given in Table 1.

The measurements at each location have been performed with the same generic mooring design, which consisted of one surface mooring and one seabed mooring. Based on experiences from the pilot phase at Location 1, the mooring design was changed before the main phase of measurements commenced. The surface mooring consisted of a surface buoy measuring surface waves and near-surface current speed (*Cs*) and direction (*CsDir*). The seabed mooring consisted of *Cs* and *CsDir* measurements throughout the entire water column and near seabed. Sea temperature and salinity were also measured. A schematic outline of the mooring configurations and instrument types are given in Figure 2.

The waves were measured every 30 minutes and the currents were measured every 10 minutes. All measured data were transferred in real-time by satellite.

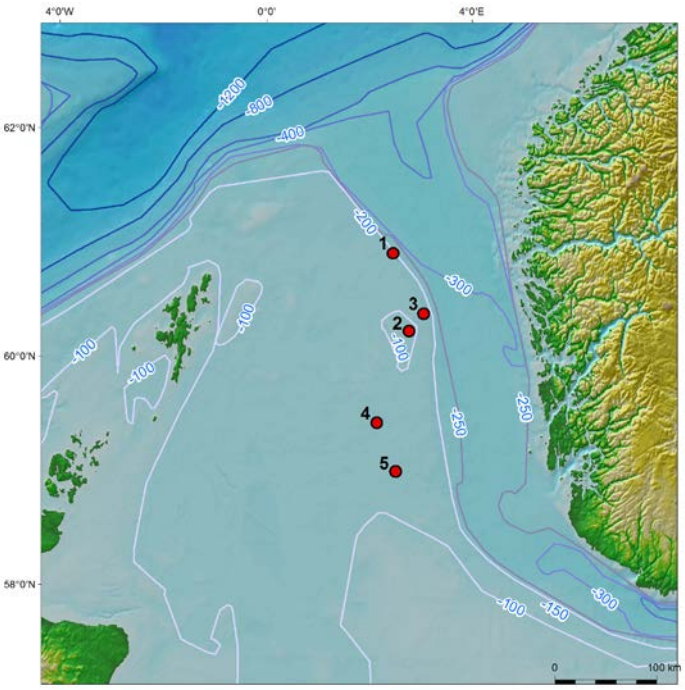

Figure 1. Measurement locations in the northern North Sea.

3 DATA QUALITY OF MEASURED CURRENT DATA

During post processing of the measured current data basic routine quality checks were applied. Nevertheless, the upper levels of the current data measured by the upward looking current profiler placed in the seabed mooring (Teledyne RD Instruments 150kHz Quartermaster ADCP, QM ADCP) contained fluctuations in subsequent measured 10-minutes *Cs*. This resulted in large spikes in the data, as illustrated in the upper panel of Figure 3. These fluctuations were too large to be real variations in *Cs* from one 10-minutes interval to the next and were thus considered to be noise in the measured current data. The spikes were most apparent down to between 40 m and 50 m water depth. Traces of this noise were also seen in measured data at larger water depths, but clearly decreasing with increasing water depths. This may be taken as an indication that the noise in measured near-surface current data was related to waves. However, filtering of the data by applying a 70-minutes running mean improved the quality in terms of reduced noise/spikes in the data and was implemented as part of the quality control. An example of time histories of *Cs* without and with a 70-minutes running mean applied is shown in Figure 3.

Discrepancies were observed between overlapping current data, i.e. *Cs* measured at the same water depth by the downward-looking current profiler placed in the hull of the surface buoy (Nortek 600 kHz Aquadopp, AQD) and the QM ADCPs placed in the seabed moorings. This is illustrated in the lower panel of Figure 3, where a time history extract from 12th to 19th of August 2014 of *Cs* measured by the two instruments at 30 m water depth at Location 4 is shown. Measured significant wave height (*Hs*) is also given.

This short time history extract captures some important features of the dominating current conditions at Location 4 and also the observed discrepancy between the current measurements. During the first three days, i.e. 12th to 15th of August, the measured *Cs* by the two different current meters corresponded quite well. Regular oscillations in *Cs* and large values of *Cs* up to nearly 60 cm/s are observed, believed to be so-called inertial oscillations. In the same period, *Hs* decreased from 4 m to around 2 m. During the next days, i.e. 15th to 18th of August, the inertial oscillations were disturbed and although measured *Cs* was less than 30 cm/s, deviations in the measured *Cs* were seen clearly. *Hs* was also low and varied around 2 m. The last day, i.e. 18th of August, the measured *Hs* increased from 2 m towards 6 m. Large deviations were seen in measured *Cs*, with the *Cs* measured by the upward-looking QM ADCP significantly lower than the *Cs* measured by the downward-looking AQD. However, the wave conditions alone cannot explain all the differences seen in this time history extract of measured current data by the AQD and the QM ADCP, as the discrepancies were also evident when *Hs* was low.

As a preliminary, preventive measure until more insight is acquired, the *Cs* and *CsDir* data measured by the AQD and by the QM ADCP from 10 m to 40 m water depth are not considered to have a sufficient quality to be included in any further analyses. Thus, only current data measured by the QM ADCP from 40 m and below and the current data measured near seabed (Teledyne RD Instruments 1200 kHz Workhorse ADCP, WH ADCP) from all available water depths have been considered for analysis. A 70-minutes running mean was applied to these measured data. This approach to the measured current data might change as new insight is gained through further investigations. Additional considerations of the quality of the measured current data can be found in Bruserud and Haver [36].

Motivated by the amount of noise seen in the QM ADCP data and the discrepancies found between the current measured by the AQD and QM ADCP, another current measurement project, called Current Verification Study (CurVeS), was initiated. To date, CurVeS consists of three different phases. For a more thorough description of all phases of and the obtained results in CurVeS and consideration of the current measurement data quality, see Bruserud and Haver [36].

The first phase of CurVeS was carried out during 2014, where the over-all aim was to compare *Cs* and *CsDir* data from multiple instruments to provide recommendations for optimal current measurements. Another important aspect was to assess the quality of the measured data of the on-going metocean measurement programme and to quantify the uncertainties prior to further analyses of these data. The new measurements were undertaken in conjunction with the on-going measurements at Location 4. Close to the existing seabed mooring another mooring was deployed for around 2 months. This mooring contained an upward-looking current profiler near seabed (Teledyne RD Instruments 75 kHz Long Ranger ADCP, LR ADCP) and three single-point current meters placed at 20 m, 30 m and 100 m water depths (Aanderaa Recording Current Meters 7, RCM7). In addition to the AQD already deployed in the hull of the surface buoy, the existing surface mooring was equipped with another downward-looking AQD deployed in a modem cage (Nortek 400 kHz Aquadopp, suspended AQD). The deviations between different current meters measuring the *Cs* at the same location and water depth were found to be much larger than expected, especially at 30 m. Thus, no clear recommendation on how current measurements best could be performed was possible to make.

To continue to assess the performance of different current instruments, a natural supplement to the first phase of CurVeS was a second phase where existing current data collected by different acoustic and mechanical instruments at the same time and location were investigated and compared. This desk study was carried out by the Norwegian Deepwater Programme (NDP) and is confidential to NDP's members, but the executive summary has been released for reference. These data have been collected at different worldwide locations, water depths and environmental conditions, but common for all the measured current data sets is that one of the current meters compared is the RPS Metocean CM04 (CM04). The main finding of the study was that "differences in observed *Cs* are usually much larger than the specified accuracies of the instruments, suggesting that the accuracy achieved in the field are often much less than the user might expect". Measured current data from two CM04s at the same location and water

depth were also compared and "showed very good agreement".

Motivated by the very good agreement found between the two CM04s at the same location and water depth in the second phase of CurVeS, a third phase of CurVeS was started in October 2015. A new mooring was deployed at Location 4, with the same design and instruments (AQD, QM ADCP and WH ADCP) as during the main measurement programme. In addition, the mooring was equipped with two CM04 deployed at 50 m and 90 m water depths. These measurements were completed in March 2016, i.e. a total duration of 6 months. The CM04 deployed at 50 m water depth did not work at all during the measurement period and no comparison between the Aquadopp and the CM04 could be made. The QM ADCP only worked for 6 days during this 6 months' period and sufficient data for a proper comparison of the measured current data by the CM04 and QM ADCP were not available. Thus, no additional knowledge can be gained through this phase of CurVeS either.

During the last decade, there has been an increasing focus on current conditions for design of offshore structures at the Norwegian Continental Shelf (NCS). Correspondingly, the way of performing current measurements has improved. In the early 1980ties, current measurements were typically performed during a couple of months at a few different water depths. Today, the state-of-the-art current measurements to be utilized in design is at least one year of current measurements through the entire water column. As current measurements are expensive to perform and thus in most cases proprietary, comparative studies focused current measurements are rarely published and relatively few, i.e. very little information about the quality and uncertainties of measured current data is available. Prior to this metocean measurement programme and all three phases of CurVeS, the possibility of such large discrepancies between different current meters and/or profilers, supposed to measure the same *Cs* at the same water depth and location, were not anticipated. In contrast to these findings, the few previous, comparable studies of overlapping current measurements performed with different current meters and/or profilers, reviewed in Bruserud and Haver [36], all reach the same conclusion; different current meters and/or profilers measuring *Cs* at the same location and water depth compare well.

Table 1. Data overview of current measurements made by the QM ADCP at each location.

Location	Water depth [m]	Data coverage					
		2011	2012	2013	2014	2015	Total, [%]
1	190						79
2	100						88
4	118						92
5	125						89

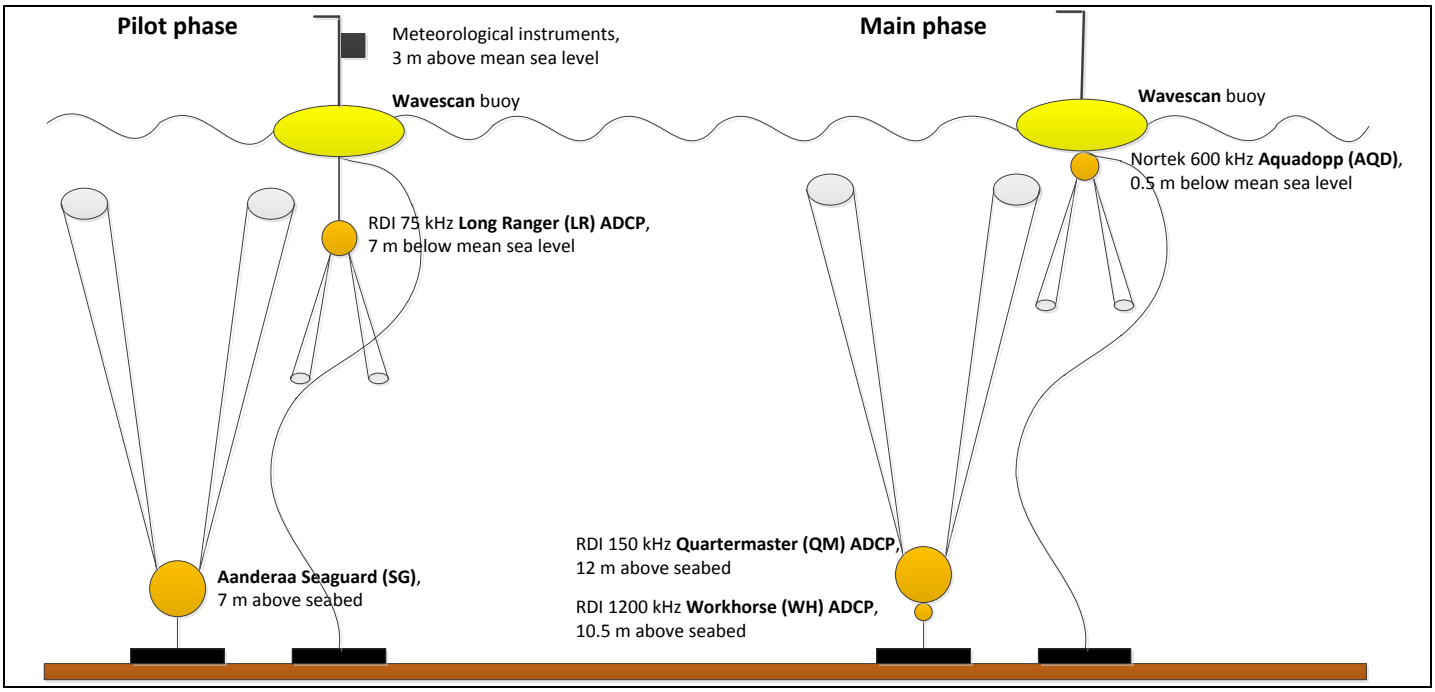

Figure 2. Schematic outline of mooring configurations and instrument types for the pilot phase at Location 1 (left) and main phase at all locations (right).

At present, no obvious or plausible explanation for such an amount of noise in the QM ADCP data and the discrepancies between the current measured by different current meters can be offered. Additional efforts are required to gain more knowledge on (I) how current can be measured more accurately for design of offshore structures and (II) how the uncertainties of measured current data can be addressed. For further investigations of the actual current measurements, it would be appropriate to do a more extensive and systematic assessment of mooring configurations, i.e. surface compared to sub-surface moorings, instrument types, i.e. acoustic compared to mechanical current meters, and sampling intervals, i.e. 10-minutes compared to a longer time interval. A natural supplement would be to review how the data quality control of the measured current data is done and also to consider to correct the measured current data for wave orbital velocities. Before more detailed knowledge about the limitations of the available current meters and some sort of specific measure of the uncertainty in measured current data are available, it would be difficult to account for the uncertainties in measured current data in design of offshore structures. However, uncertainties like these are important to be aware of and consider in design of offshore structures, but how such uncertainties best could be implemented in analysis of current speed data still remains to be determined.

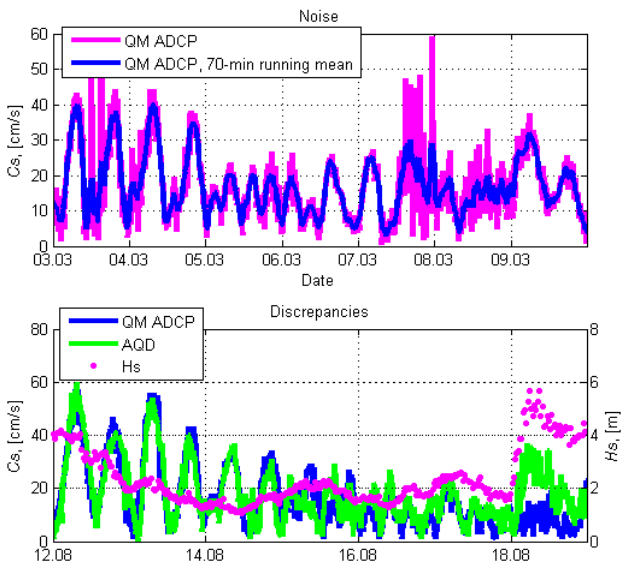

Figure 3. Illustration of the data quality issues related to measured current data; "noise" in measured current data before filtering compared to filtered current data (upper panel) and discrepancies observed between overlapping current data (lower panel).

4 VARIATIONS IN CURRENT CONDITIONS

In addition to the anticipated investigation of joint distribution of waves and currents, the extensive current data set from the metocean measurement programme can also be used to describe, investigate and gain further knowledge about the current conditions in this part of the northern North Sea.

4.1 Spatial variations

At all the four locations, the directional distribution of measured *Cs* varies very little with water depth and the *Cs* decrease with increasing depth. At Location 1 most of the currents is toward a south-easterly direction, whereas the dominating *CsDir* at the other locations is towards south. At Location 4 and 5 currents towards north are also prominent. Thus, the directional distribution at Location 1 stands out from the three other locations. As Location 1 is further north and thus not sheltered by the Shetland Islands for Atlantic inflow to the same extent as the three other locations further south, see Figure 1, this can explain the observed difference in directional distribution of currents. In addition, Location 1 is in an area with steeper bottom topography and larger water depths, i.e. the westside of the Norwegian Trench, and in such areas currents are known to follow the bottom topography.

Regarding maximum *Cs*, this is found to be larger at Location 1 than at the three other locations. The maximum *Cs* at Locations 2 and 4 are similar, but the maximum *Cs* at Location 5 are significantly larger. The largest *Cs* at Location 5 are caused by an episode of large *Cs* during 24th and 25th of December 2012. During the same period, *Cs* are less than 20 cm/s at the other locations. This indicates a spatial variation in current conditions, also for locations near each other and with approximately the same water depth.

4.2 Seasonal variations

Seasonal variations in both *Cs* and *CsDir* have also been investigated. In general, the magnitudes of mean and maximum *Cs* at all water depths are largest during winter, decrease in the spring, are lowest during summer and increase again in the autumn. However, at Locations 4 and 5 the seasonal maximum *Cs* in the summer is larger than in the spring and autumn. The estimated extreme *Cs* values follow this trend. The reason for this is two episodes with large *Cs*, in August 2011 and 2014, respectively. Time series of *Cs* at 40 m, 80 m and 3 m above the seabed at Locations 4 and 5 during the latter of these episodes are shown in Figure 4. As seen in Figure 3, regular oscillations in *Cs* with large *Cs* values, believed to be inertial oscillations, are seen. At both Locations 4 and 5, relatively large wind speeds in the range 15 m/s to 25 m/s and a change in wind direction are observed just before the oscillations in *Cs* are initiated. The magnitude of *Cs* of inertial oscillations is essentially controlled by the depth of the mixed layer. During summer and autumn when the mixed layer is relatively thin, currents associated with inertial oscillations can be reasonably large. Thus, it is not surprising that inertial oscillations generating large *Cs* are observed in August. The *Cs* values during these two episodes are larger at Location 5 than at Location 4.

At all locations, the *CsDir* vary very little with season. In accordance with the annual directional distributions, the *CsDir* at all locations shows only marginal variation

between 40 m and 80 m water depth, but some variations are seen near seabed.

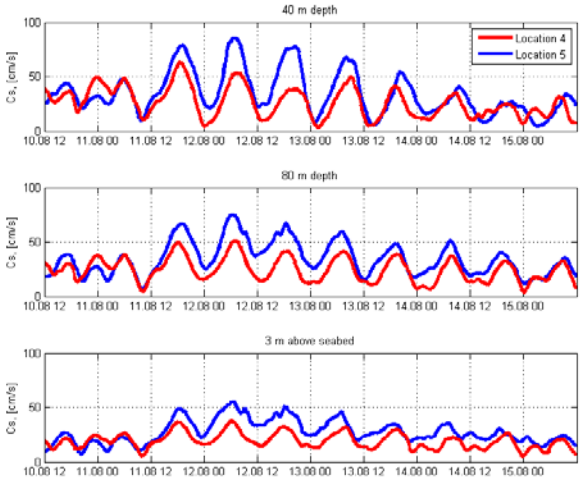

Figure 4. Time series of *Cs* at Location 4 and 5 during one of the two episodes with large *Cs* in August 2014.

4.3 Inter-annual variations

To study the year-to-year variability of extreme *Cs*, the extreme *Cs* have been estimated for each individual year of current measurements. In Table 2 the estimated *Cs* with annual probability of exceedance 10^{-1} and 10^{-2} are given at 40 m water depth at all four locations for each year. In Figure 5 an illustration of the variation in estimated *Cs* with annual probability of 10^{-2} based on each individual year of measurements is shown.

The year-to-year variability of extreme *Cs* is large at all locations. In general, the largest estimated extreme *Cs* are found in 2011 and the smallest in 2013. At Location 1, the difference between the largest and smallest *Cs* with annual probability 10^{-2} is approximately 20 – 25 cm/s for the different water depths, i.e. the between 20 and 35% (for the smallest relatively to the largest *Cs* with annual probability 10^{-2}). Comparable numbers are found for Location 2. Both at Locations 4 and 5, this difference is seen to be even larger. At Location 4, the difference is approximately 40 – 50 cm/s, i.e. between 30 and 50 %, while even larger at Location 5 where the difference is approximately 65 – 80 cm/s, i.e. over 50 %.

Previously, very little inter-annual variability in current conditions has been anticipated. Based on that assumption, only one year of current measurements is recommended when extreme current conditions for design are to be established. Current measurements lasting for more than one year are rarely available. The shown year-to-year variations in estimated extreme current conditions suggest large uncertainties when based on one year of current data only. This uncertainty can go in both directions; the estimated extreme current conditions may be conservative or, more important, non-conservative. To account for the observed year-to-year variability and reduce this uncertainty and thus obtain more robust estimates of extreme current conditions, more extensive current data covering several years, are necessary when extreme current conditions for design are to be estimated. However, considering the discussed uncertainties in measured current data, performing current measurements for several years might not be the ideal either.

According to Bruserud and Haver [37], current hindcast of good quality have recently been developed for the northern North Sea. It is pointed out that the quality of this current hindcast is not as good as the quality of the available wind and wave hindcast for the NCS and must be used with caution. Nevertheless, this constitute a very promising starting point for further development of an even better current hindcast for the northern North Sea. Rather than performing current measurements for several years, development of high-quality current hindcast covering several years, validated with a shorter period of current measurements, could prove to be a more appropriate and prosperous approach to obtain more reliable estimates of extreme current conditions for design. In order to have any confidence in such an approach, the problem of how to perform high-quality current measurements with well-defined uncertainty bands still remains to be solved.

Table 2. Extreme values for year-to-year *Cs* at 40 m water depth at all four locations.

Location	Period	Annual probability of exceedance, [cm/s]		
		0.63	10^{-1}	10^{-2}
1	2011	104.2	115.2	128.4
	2012	88.6	97.7	108.5
	2013	87.1	96.0	106.5
	2014	90.9	100.9	113.0
	2015	107.4	117.6	129.5
2	2011	88.8	99.4	112.2
	2012	78.5	86.1	95.1
	2013	66.6	72.4	79.3
	2014	76.5	83.9	92.7
	2015	68.4	74.7	82.3
4	2011	61.6	68.2	76.0
	2012	84.3	96.0	110.5
	2013	49.3	53.6	58.6
	2014	64.7	71.5	79.8
	2015	57.2	62.7	69.1
5	2011	88.1	99.7	113.9
	2012	113.7	132.4	156.1
	2013	62.7	69.1	76.6
	2014	104.9	120.7	140.3
	2015	88.6	100.4	114.8

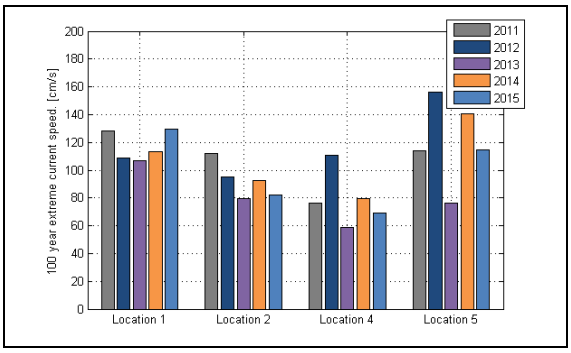

Figure 5. Variation in 100 year extreme *Cs* based on individual years at 40 m water depth at all four locations.

5 CASE STUDY

In lack of more detailed and verified joint models of metocean actions, N-003 Edition 3 [1] recommends a conservative approach to combination of metocean processes (see section 10.3, Table 7). However, the degree of conservatism is not very well known. Two case studies have been performed on different combinations of wave and currents to estimate characteristic metocean loads, i.e. ULS load, on a jacket. Both case studies are based on the same simplified load model, but the first comprised simplified metocean data and the other was based on the measured wave and current data from the northern North Sea.

5.1 Methodology

Load estimation

For a jacket, the governing load process is the hydrodynamic load caused by waves and currents. A simple parametric model for overturning moment of a jacket, which neglects the effect of dynamics, was developed by Heideman [38]. The model can be used to estimate a generic, static load (overturning moment), *L [MN]*, on a jacket and is given as

$$L = K_1(H + K_2 Cs)^{K_3} \tag{1}$$

where H is individual wave height [m], Cs is depth integrated current speed [m/s] and K_1, K_2 and K_3 are empirical constants. For a drag dominated jacket platform in about 100 to 200 m water depth, the following parameters are expected to give reasonable quasi static loads; K_1 set to 0.03, K_2 set to 5.5 and K_3 set to 2.2 [39]. It is obvious from the empirical constants that the waves will be of most importance for the loads.

Two main approaches were considered when the ULS load was to be estimated:

1. ULS load estimated according to N-003 (see section 10.3, Table 7, [1]) where H of annual probability of exceedance 10^{-2} and Cs of annual probability of exceedance 10^{-1} are combined in the load model given in Equation (1) to obtain the ULS load. Since waves are most important for the estimated loads and will result in the largest loads, the combination of H of annual probability of exceedance 10^{-1} combined with Cs of annual probability of exceedance 10^{-2} is not considered.

2. ULS load estimated directly from a load time series where (1) time series of H and Cs were combined by Equation (1) into a time series of the load, (2) a probability distribution was fitted to the load time series and (3) the extreme load with annual probability of exceedance 10^{-2} was estimated.

ULS load estimated according to N-003

For estimation of the extreme H both the all-sea states (initial distribution) and peak-over-threshold approaches were considered.

For the all-sea states approach, the long-term distribution H during T hours is given by

$$F_H(h) = \iint_{hs\,tp} F_{H|Hs,Tp}(h|hs,tp) f_{HsTp}(hs,tp)\,dhs\,dtp \tag{2}$$

where

$F_{H|Hs,Tp}(h|hs,tp)$ is the short-term distribution of H during T hours based on Forristall's distribution for individual waves [40]:

$$F_{H\S Hs,Tp}(h|hs,tp) = \left(1 - exp\left[-2.263\left(\frac{h}{Hs}\right)^{2.126}\right]\right)^{\frac{T\cdot 60\cdot 60}{0.77Tp}} \tag{3}$$

and

$f_{HsTp}(hs,tp)$ is long-term variation of wave climate described by the joint probability density distribution for Hs and Tp based on a lognormal-Weibull distribution for Hs and a lognormal distribution for $Tp|Hs$ [8].

For the peak-over-threshold approach, according to the method proposed by Tromans and Vanderschuren [41], the long-term distribution of storm maximum wave heights, H_M, in a random storm can be given by

$$F_{H_M}(h) = \int_{\tilde{h}} F_{H_M|\tilde{H}}(h|\tilde{h}) f_{\tilde{H}}(\tilde{h})\,d\tilde{h} \tag{4}$$

where

$\tilde{H}$ is the most probable maximum storm wave height given in a specific storm by given by, see for instance [42],

$$F_{\tilde{H}|storm}(\tilde{h}|storm) = \prod^{i} F_{H_i}(\tilde{h}) = 0.37 \tag{5}$$

where i is the number of storm steps exceeding the selected threshold of storm peak Hs and F_{H_i} is the short-term distribution of H during each storm steps i, given by Forristall's distribution for individual waves (Equation (3)). $\tilde{H}$ is estimated for all the storms with storm peak Hs exceeding the selected threshold of Hs. The long-term distribution of $\tilde{H}$, $f_{\tilde{H}}(\tilde{h})$, is assumed to be well modelled by a Weibull 3-parameter distribution.

and

$F_{H_M|\tilde{H}}(h|\tilde{h})$ is the short-term distribution of H_M, given in terms of a new variable $V = \frac{h}{\tilde{h}}$ assumed to follow a Gumbel distribution

$$F_{H_M|\tilde{H}}(h|\tilde{h}) = exp\left(-exp\left[-\left(\frac{\left[\frac{h}{\tilde{h}}\right]-1}{\beta_v}\right)\right]\right) \quad (6)$$

The extreme *Cs* can be estimated the traditional way by fitting a 3-parameter Weibull distribution to all the measured current data.

<u>ULS load estimated directly from a load time series</u>
Before a load time series can be calculated from Equation (1), a time series of *H* must be calculated based on the short-term sea state, i.e. the *Hs* and *Tp* data. Two approaches are chosen, both based Forristall's distribution for individual waves (Equation (3), please note that for simplicity $F_{H|Hs,Tp}(h|hs,tp)$ is shortened to $F_H(h)$ and *T* ·*60·60/0.77Tp* to *n*):

1. <u>The most probable value of *H*</u>
   When Equation (3) is inverted, *H* will be given as

$$H = Hs\left[-\frac{1}{2.263}\left(ln\left[1-F_H(h)^{\frac{1}{n}}\right]\right)\right]^{\frac{1}{2.126}} \quad (7)$$

   H will have its most probable value when $F_H(h)$ is approximately 0.37 [42]. The short-term variability of *H* within the sea state is neglected.

2. <u>Monte Carlo simulated *H*</u>

$$H = Hs\left[-\frac{1}{2.263}\left(ln\left[1-R^{\frac{1}{n}}\right]\right)\right]^{\frac{1}{2.126}} \quad (8)$$

   where *R* is a random number between 0 and 1, representing a possible realization of $F_H(h)$. This is done for every time step of the wave data and a possible time series of *H* is established. Then the procedure is repeated 100 times and hence 100 different possible time series of *H* are established. This approach is a way to include short-term variability in *H*.

Time series of the load, *L*, can be estimated based on Equation (1) with the different time series of *H* and *Cs*. The long-term distribution of *L* and corresponding extreme values are modelled in terms of a 3-parameter Weibull distribution. For the load time series based on Monte Carlo simulated *H*, 3-parameter Weibull distributions are fitted to the 100 different realizations of the time series of *L* and a mean value of the 100 Weibull parameters and corresponding estimated extreme loads are given.

For further details on the methodology of the case study, the reader is referred to Bruserud and Haver [43].

5.2 Case study - example location

Bruserud and Haver [43] indicated the possible conservatism in the Norwegian design regulations for estimation of quasi-static metocean loads on a jacket based on the methodology described in section 5.1. In addition, the effect of a longer time step for current data than the standard 10-mintues *Cs*, i.e. 3 hours as for wave, was studied. The following three time steps for the current data were defined

- **10-min *Cs*;** the standard time interval current measurements are performed, i.e. 10 minutes
- **3-hrs mean *Cs***; averaging all 18 measured 10-minutes *Cs* values during 3 hours
- **3-hrs max *Cs***; selection of the largest 10-minutes *Cs* value during 3 hours

Thus, the possible variations in the load estimation according to N-003 based on different approaches to the data and data analysis were assessed.

The wave and current data utilized in that study was from a deep-water location on the Norwegian Continental Shelf where good quality measured current data was available for the longest period. Assumptions were made, so the current data could be adjusted and made representative for current conditions at a typical jacket location. For waves, hindcast data from the Norwegian reanalysis archive (NORA10) [44] was used. The time step for wave data was 3 hours and for current data 10 minutes.

Table 3 summarizes the most important results from this sensitivity study for this example location [43] and gives the different estimated ULS loads, normalized to ease comparison. When the extreme waves and currents are estimated based on a peak-over-threshold approach, the corresponding estimated ULS load is reduced compared to all-sea states approach. The reduction is approximately 10 %. When a 3-hrs averaging of *Cs* is utilized, the estimated ULS load is somewhat reduced. This is seen both for the all-sea states and peak-over-threshold approaches. The reduction is of similar size, typically a few percent. As expected, the 3-hrs mean *Cs* gives a slightly smaller ULS load than the 3-hrs max *Cs*. When the ULS load is estimated directly from a time series of the load, the ULS load is reduced significantly. Uncertainties apply to this result since the effective length of the joint wave and current data and hence the load time series, is 47 months.

Table 3. Normalized ULS load according to N-003 and directly from a load time series.

Location	Waves	Current	ULS load
Example	All-sea states *H*	10-min *Cs*	1.00
		3-hrs mean *Cs*	0.95
		3-hrs max *Cs*	0.97
	Peak-over-threshold *H*	10-min Cs	0.89
		3-hrs mean *Cs*	0.86
		3-hrs max *Cs*	0.88
	Most probable *H*	10-min *Cs*	0.54
		3-hrs mean *Cs*	0.52
		3-hrs max *Cs*	0.56
	Monte Carlo *H*	10-min Cs	0.62
		3-hrs mean *Cs*	0.60
		3-hrs max *Cs*	0.64
1	All-sea states *H*	3-hrs max *Cs*	1.00
	Most probable *H*		0.66
2	All-sea states *H*	3-hrs max *Cs*	1.00
	Most probable *H*		0.73
4	All-sea states *H*	3-hrs max *Cs*	1.00
	Most probable *H*		0.64
5	All-sea states *H*	3-hrs max *Cs*	1.00
	Most probable *H*		0.61

5.3 Case study - northern North Sea

Another case study was performed where the load model described in section 5.1 is applied to estimate the ULS loads based on the recent wave and current measurements at the four locations in the northern North Sea. The water depths at these locations are approximately 100 m, 120 m, 130 m and 190 m (see Table 1), i.e. typical water depths where jackets would be placed.

Based on the results from the case study at the example location (previous section), only one sensitivity case was selected for each of the two main approaches for estimation of ULS load

- according to N-003 with an all-sea states approach for estimation of extreme values and 3-hrs max *Cs*
- directly from a time series of the load based on most probable *H* for each 3-hour sea-state, i.e. neglecting short-term variability within the sea-state, and 3-hrs max *Cs*

The parameters for the joint distribution of *Hs* and *Tp* and corresponding estimated extreme values for *H* are given in Table 4 and the Weibull parameters and the corresponding extreme values of *Cs* are given in Table 5.

Estimated values of the ULS load according to N-003, based on the estimated extreme *H* and *Cs*, are given in Table 3. The Weibull parameters and corresponding extreme values of *L* estimated directly from a time series of the load are given Table 6. Please note that the values have been normalized to ease comparison.

At all the four locations, when these two approaches for estimation of ULS load are compared, a significant reduction in estimated ULS load directly from a time series is seen, ranging from around 25 % to 40 %.

Uncertainties apply to the results presented here, mainly due to the length of the measured current data. However, these results are considered to give a reliable indication of the ULS load when sufficient joint data are available; i.e. reductions in ULS load.

Table 4. Parameters for the joint distribution of *Hs* and *Tp* and estimated extreme values for *H* [m].

Location	Distribution	Parameters					Annual probability of exceedance, [m]		
							0.63	10^{-1}	10^{-2}
1	**lognormal-Weibull for *Hs***	γ	β	α	ϑ	θ	21.9	25.3	28.8
		2.763	1.534	0.553	3.941	0.862			
	lognormal for *Tp|Hs*	a_1	a_2	a_3	b_2	b_3			
		5.193	-1.857	0.219	0.299	-0.046			
2	**lognormal-Weibull for *Hs***	γ	β	α	ϑ	θ	19.3	22.2	25.1
		2.822	1.658	0.553	3.436	0.862			
	lognormal for *Tp|Hs*	a_1	a_2	a_3	b_2	b_3			
		2.377	0	8.314	0.220	-0.225			
4	**lognormal-Weibull for *Hs***	γ	β	α	ϑ	θ	19.6	22.7	25.8
		2.800	1.603	0.553	3.646	0.862			
	lognormal for *Tp|Hs*	a_1	a_2	a_3	b_2	b_3			
		13.956	-11.395	-0.003	0.135	-0.003			
5	**lognormal-Weibull for *Hs***	γ	β	α	ϑ	θ	19.1	22.1	25.0
		2.819	1.648	0.553	3.470	0.862			
	lognormal for *Tp|Hs*	a_1	a_2	a_3	b_2	b_3			
		4.320	-1.670	0.021	0.103	-0.050			

Table 5. Weibull parameters and corresponding extreme values for *Cs* [cm/s].

Location	Weibull parameters			Annual probability of exceedance		
	γ	β	α	**0.63**	10^{-1}	10^{-2}
1	1.450	18.28	6.79	86	98	112
2	1.579	15.79	6.17	67	75	85
4	1.323	10.06	8.14	59	67	76
5	1.065	9.52	9.24	80	94	112

Table 6. Weibull parameters and corresponding extreme values for *L* [MN].

Location	Weibull parameters			Annual probability of exceedance		
	γ	β	α	**0.63**	10^{-1}	10^{-2}
1	0.741	1.51	0.53	27.40	35.59	46.58
2	0.661	0.79	0.30	20.21	27.14	36.74
4	1.507	6.32	-2.54	23.49	27.13	31.39
5	0.668	0.74	0.23	18.26	24.46	33.02

6 CLOSING REMARKS

Motivated by the potential in simultaneous metocean data for design of offshore structures and to acquire data of sufficient length to establish joint distributions for waves and currents, a metocean measurement programme of waves and current profiles at five locations in the northern North Sea has been performed for nearly five years.

The main experiences and most important learnings from the metocean measurement programme can be summarized as follows:

- **Quality of current measurements**
  Despite quality control, the accuracy of the measured current data was found to be less than expected. Differences in measured current speeds were much larger than the specified accuracy of the instruments. Despite efforts to improve knowledge on different methods and currents meters to perform current measurements for design of offshore structures, further work is required to address and account for the uncertainties of the measured current data.

- **Governing current conditions**
  At Location 2, 4 and 5 inertial oscillations have been observed in the measured current data and found to generate many of the largest observed current speeds. Thus, inertial oscillations seem to be the governing current conditions in this part of the northern North Sea.

- **Operational current conditions**
  In general, the expected seasonal variation in current speeds with the current speeds largest during winter, decreasing during spring and summer before increasing during the autumn, is confirmed. However, at Locations 4 and 5, the maximum current speeds are larger in the summer than the spring and autumn, due to inertial oscillations during the summer.

- **Extreme current conditions for design**
  The variation in estimated extreme current speeds based on different individual years of data is large and larger than assumed previously. This suggest that current measurements of longer duration than one year will give more reliable estimates of extreme current conditions for design of offshore structures.

- **Potential in simultaneous wave and current data**
  When the ULS load for a jacket is estimated directly from a time series of the load, the ULS load is reduced significantly compared to the more traditional approach according to N-003. Although the results are intended to be illustrative and not suitable for design, these results are considered to give a reliable indication of the ULS load when sufficient simultaneous metocean data are available.

The most important experience from this metocean measurements programme is considered to be the new insight regarding the accuracy of the measured current data. No additional knowledge was gained through the three phases of the Current Verification Study, aimed to give guidance on how the accuracy of current measurements could be improved and further work is necessary. Both good quality and sufficient length of simultaneous metocean data are required to establish joint distributions. If either of the wave and current data, in this case the measured current data, do not have adequate quality, it will not be appropriate to establish joint distributions of simultaneous data as the reliability of such will be deteriorated by the data quality.

To utilize that the occurrence of extreme wind, waves and currents are not fully correlated in design of offshore structures, Norwegian design regulations presently recommends at least three years of simultaneous wind, wave and current data. For wind and waves, both measured and hindcast data are of sufficient quality and length. For currents, measured current data has mainly been used and none of the available current hindcast for the Norwegian Continental Shelf (NCS) are considered to hold the required quality. Considering the quality of measured current data presented and discussed in this article, it might be more appropriate and prosperous to develop alternatives to measured current data for design of offshore structures and utilization of simultaneous occurrence of metocean parameters. Rather than to measure current simultaneously with wind and waves for a long period, development of high-quality current hindcasts, validated with a shorter period of current measurements, could prove to give more reliable estimates of extreme current conditions for design of offshore structures. However, to have any confidence in such an approach, the challenge of how to perform high-quality current measurements with well-defined uncertainty bands still remains to be solved.

ACKNOWLEDGMENTS

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3.2 Presentation & article by Sverre Haver

Airgap and Safety:

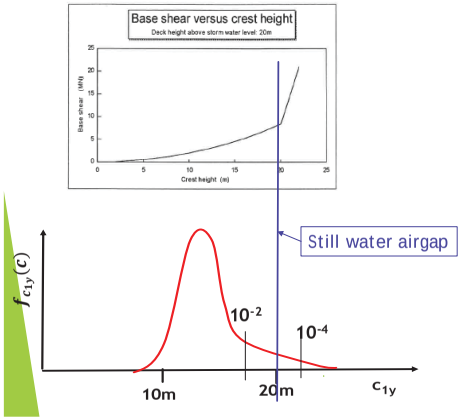
Metocean Induced Uncertainties Affecting Airgap Assessment

Sverre Haver, UiS/NTNU

Rule Regimes for Offshore Structures in Norwegian Waters

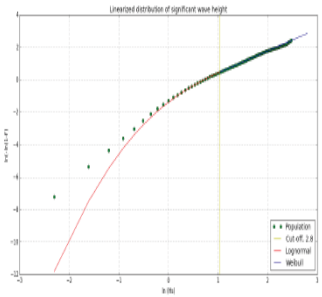
PSA Regime	Maritime Regulation
▶ Framework Regulation ➔ Facility Regulation ➔ Norsok Standards ▶ For an annual probabilty of 10^{-2} or higher, waves shall not hit areas where crew may be in extreme weather. ▶ Structure shall widthstand accidental crest height defined as the 10^{-4} - annual probability crest height. ▶ Model tests is recommended if results are associated with significant uncertainties.	▶ Framework Regulation ➔ Maritime Regulation (Red Book of Norwegian Maritime Directorate) ➔ Classification Rules from e.g. DNV-GL. ▶ For an annual probabilty of 10^{-2} or higher, waves shall not hit areas where crew may be in extreme weather. ▶ No (or vague) requirments regarding robustness. ▶ No recommendation of model tests, but if a model test is available, it shall be used for validation of results.

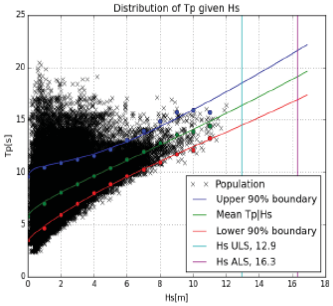
Airgap and Safety



- The distribution function for annual maximum crest height (or relative crest height for floaters) is associated with uncertainties.
- If submergence of deck level becomes too large a fixed platform may collapse.
- For a floater, more or less severe local damage are more likely. In rare cases situation may escalate.
- If a severe cellar deck submergence is experienced, mooring lines may fail due to impact load.

Long term description of wave climate, $f_{HSTP}(h,t)$





Long term distribution of 3-hour maximum crest height:

$$F_{C_{3h}}(c) = \int_h \int_t F_{C_{3h}|H_s T_p}(c|h,t) f_{H_s T_p}(h,t)\,dt\,dh$$

Contributions to required airgap: Fixed Platforms

Contribution to required airgap	Jack-ups & Jacets		Gravity based structures	
	ULS	ALS	ULS	ALS
Tidal amp. (m)	1.0	-	1.0	-
Storm surge (m)	0.9	1.1	0.9	1.1
Second order point crest height (m)	15.1	19.6	15.1	19.6
Uncertainty 2. order (m)	0 – 1.6	0 - 2	0.8 +/-0.8	1 +/- 1
Area maximum (m)	1.5	2.0	1.5	2.0
Platform-Wave Interaction (m)	-	-	1.5 -4.5	2 - 6
Effect of current (m)	-	-	-	-
Margin (m)	-	-	-	-
Required airgap (m)	18.5 – 20.1	22.7 – 24.7	20.0 –24.5	24.7 – 30.7

Leading uncertainties

Wave structure interaction of fixed platforms

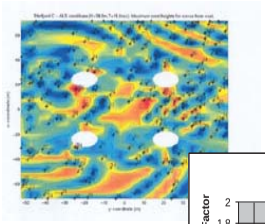
Jackets and jack-ups

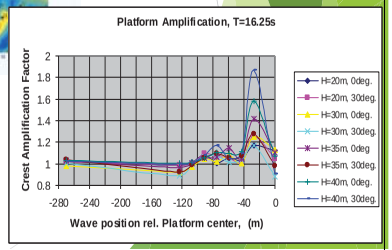
?

Possibly small

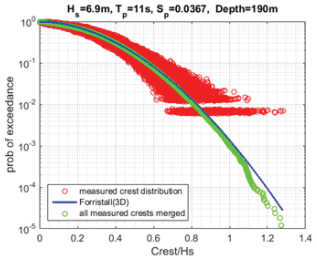
but not zero!

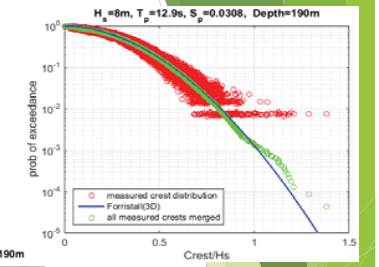
Condeeps

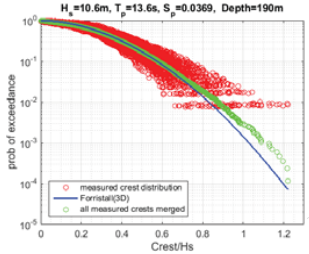




Adequacy of second order







My view:

Results from 2. order is a lower bound for the crest height

Airgap assessments semi-submersibles

- Target variable: 3-hour maximum relative crest height, i.e. point 3-hour maximum crest caused incoming second order surface process (long crested or short crested), corrected for wave - structure interaction and accounting for the simultaneous motion of deck vertically above point under consideration.
- Motion of platform depends on metocean condition for a given draft: Mean wind speed, significant wave height and spectral peak period with associated directions. These quantities define the wind spectrum and the wave spectrum.
- Wave - structure interaction is dependent significant wave height, spectral peak period and platform motions. Furthermore, the effect on the undisturbed crest height will vary in space and it will vary with incoming direction.
- A full long term analysis of relative crest height of floater will be complicated and time consuming. A more attractive approach is metocean contour lines. This is an approximate method for assessing long term extremes using short term analyses.

Assessing required still water airgap

- Analysis can be done using numerical methods. Often surface process is modelled as a stationary Gaussian process and linear approaches are used for assessing platform motions and wave structure interaction.

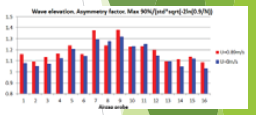
  Then:

  The disturbed crest height must be corrected to account for asymmetry in incoming real sea. Factor: 1.2

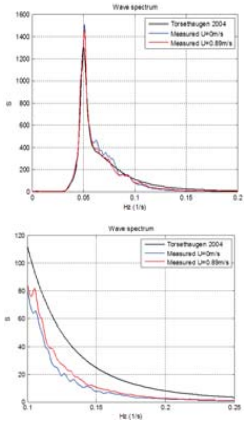
  Results cannot be trusted close (< half a diameter?) to columns.

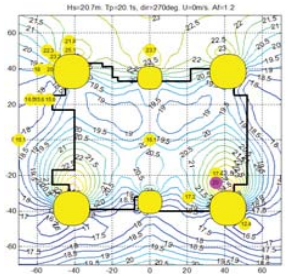
  Effects of wind must be included in an approximate way.

- Analysis is preferably done by model tests, possibly in combination with numerical analysis.



Uncertainties in model test spectrum

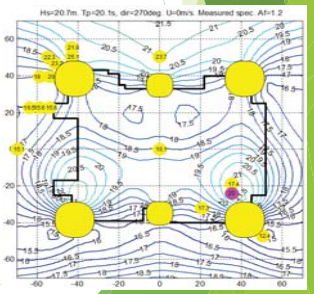




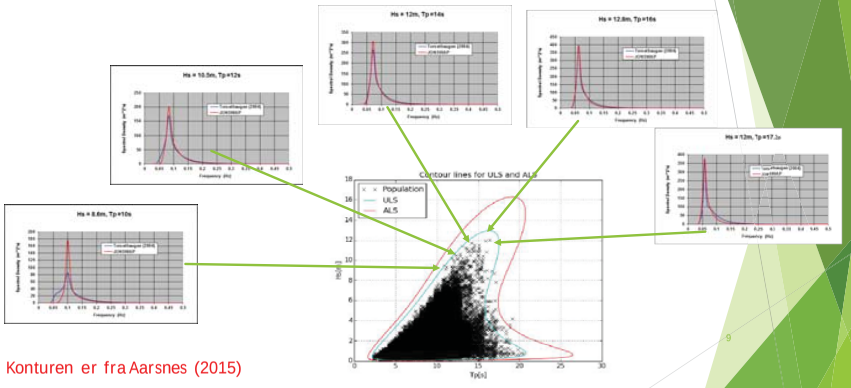
Torsethaugen

The high frequency part of wave spectrum is of some concern. As is the ability to reproduce target spectrum in tests!

«Measured» spectrum



Design sea states and associated wave spectra



Konturen er fra Aarsnes (2015)

Long Crested or Short Crested Waves

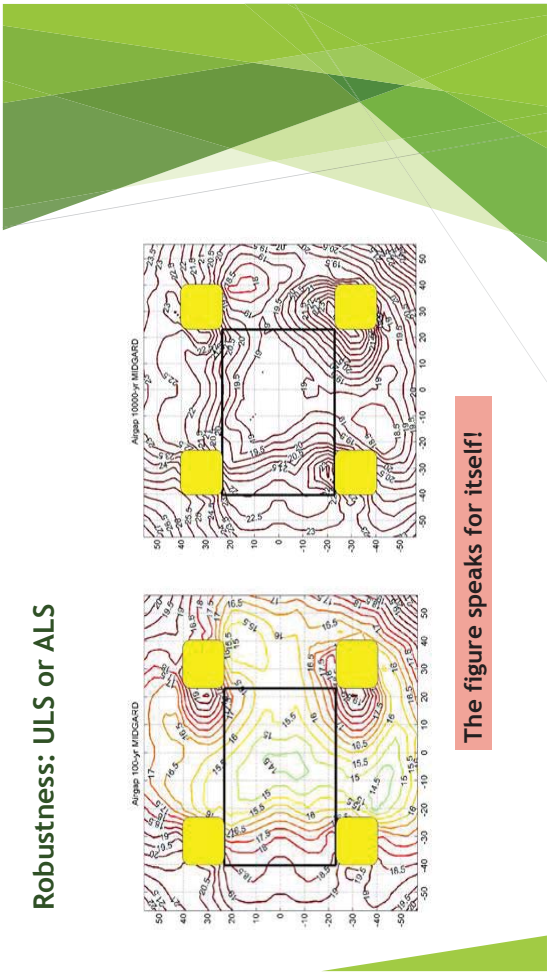
- Real sea is short crested!
- Ewans spreading function is generally adopted as a good model.
- Data underlying Ewans spreading function are for rather low sea states - say Hs < 5m?

  Is there any concern of using this for Hs > 16m?

- One thing is the average spreading of a sea state. What about the local spread for the most severe waves / wave groups relative to the average spread?

  Will the most severe wave events be less spread than the avearage spread?

  Is there any information about about crest length and associated particle speed and phase speed of the wave?


Robustness: ULS or ALS
Airgap 100-yr MIDGARD
Airgap 10000-yr MIDGARD
The figure speaks for itself!

Airgap and safety: Metocean induced uncertainties affecting airgap assessments

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Airgap and Safety: Metocean Induced Uncertainties Affecting Airgap Assessments

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ABSTRACT: The safety aspect of the airgap question is discussed. Thereafter uncertainties related to airgap assessments will be discussed both for fixed platforms and semi-submersibles. The various sources are discussed and illustrated with examples. Results are discussed both for 10^{-2} – annual probability scenario (ULS) and an ALS scenario (10^{-4} annual probability). This because the most important difference between the two possible rule regimes for Norwegian Continental Shelf is the way ALS is introduced regarding airgap.

KEY WORDS: Airgap; Second order Wave Theory; Wave Spectrum; Uncertainties.

1 INTRODUCTION

Airgap is the distance between the underside of the deck structure of a fixed or floating structure and wave crest vertically underneath the considered deck point. The aim of the airgap assessment is to validate that the smallest airgap under the deck is acceptable at the target annual exceedance probability. The floating structures considered here are semi-submersibles. Column based structures will for steep seas typically experience run-up. Close to the columns the cellar deck bottom must therefore be designed for run-up induced impacts. The run-up phenomenon will not be dealt with in this paper.

The still water airgap (airgap in absence of waves) is the quantity focused on in the design. For floaters the still water level will for airgap consideration is always equal to the calm surface level, while for fixed platform it will vary with tide and storm surge. This must be baked into the still water level in connection with airgap assessments for fixed platforms.

In practise, the aim of the airgap assessment is to determine the necessary still water airgap in view of the requirements of the adopted governing rule regime. Irrespective of platform concept, the crucial quantity is the height of the extreme crest heights. Either in terms of the extreme height of the undisturbed crest or the extreme of the crest heights disturbed by the presence of the structure. For floating structures on must additionally account for the motions of the platform.

This paper will discuss various metocean induced uncertainties and how they may affect the airgap assessment.

2 AIRGAP AND SAFETY

Why could airgap – or rather - lack of sufficient airgap affect safety? A generic drag dominated jacket is considered in Ref. [1]. The base shear is shown versus crest height in Figure 1 for a case with still water airgap of 20m. It is seen that base shear is increasing rapidly as the still water airgap is exceeded. A 2m submergence over the full width of platform is more or less doubling base shear. The exceedance probabilities for various base shear levels are shown in Figure 2. For no wave-deck impact, still water airgap is well above the most extreme crest heights, while for the other case the still water airgap is 17.5m.

The characteristic 10^{-2} -annual probability base shear is about 23MN, i.e. using a partial safety factor of 1.3 the ULS design value is 30MN. The ALS characteristic value (10^{-4} – annual probability) assuming no wave-deck impacts is about 41MN, i.e 30% larger than the ULS design value for this drag dominated generic jacket. If wave-deck impact is accounted for with specified airgap, the ALS base shear is about 66MN, i.e. the ALS base shear is increased with more than 50%. With wave-deck impact the ALS base shear is more than two times the ULS design value.

A semi-submersible could well experience mooring line failure if an extreme wave-topside impact occurs when the up-wave mooring lines is highly tensioned due to a large down-wave offset as the impact happens. Another scenario is local damage to columns and topside that can develop into a more severe situation, this scenario is possible of most interest for wave - deck impacts of floaters.

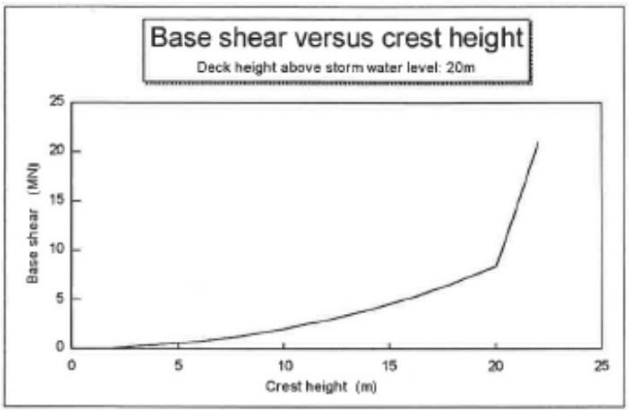

Figure 1. Generic base shear for a drag-dominated jacket with still water airgap of 20m. From Ref. [1].

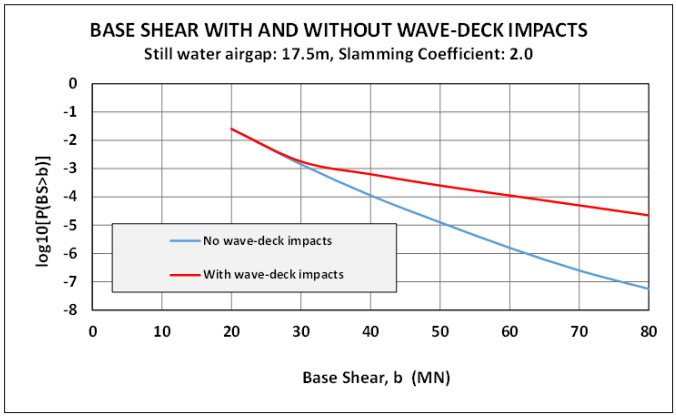

Figure 2. Annual exceedance probabilities of base shear with and without accounting for wave deck impacts. Reconstructed from Ref. [1].

3 RULE REGIMES

Two rule regimes are available for offshore structures at the Norwegian Continental Shelf, see e.g. Ref. [25]:

RR1: The Framework Regulations → The Facilities Regulations → Norsok Standards.

RR2: The Framework Regulation → Maritime Regulations. (The Red Book of Norwegian Marine Directorate) with supplementary rules provided by a classification society, e.g. DNV-GL.

For fixed platforms, RR1 will apply. For most floating installations intended to be on the field through the full operational life, RR1 is also generally applied. On the Norwegian Continental Shelf, RR2 is primarily used for floating installations operating for a limited period at a field, e.g. drilling rigs. For further details, reference is made to Ref. [25] and references given therein.

At ULS level, the basic requirements regarding airgap for the two rule regimes are not very different. For an annual probability of 10^{-2}, waves shall not hit areas where crew members can be in extreme weather situations. In both rule regimes, this is accounted for by requiring no major wave - deck impacts at a 10^{-2} - annual probability. Local impacts (e.g. impacts caused by run-up jets along columns which are difficult to avoid) can be accepted if the corresponding wave loads are accounted for in design.

Regarding airgap assessments, the major difference between the two rule regimes is how a safety margin on top of the requirement above (positive airgap at ULS level) is implemented. Beyond a positive airgap, there is no requirements for additional margins in the maritime regulation, see e.g. Ref. [2], but some more specific guidance on how to predict necessary ULS airgap have lately become available, Ref. [3], as a response to the accident with COSL Inventor in the Northern North Sea in December 2015. For RR1, Ref. [4] also requires a positive airgap at ULS level (10^{-2} annual probability), but, in contrast to RR2, it is additionally recommended to design for positive airgap at the ALS level (10^{-4} annual probability level) due to the inherent complexity of the airgap problem. The difficulty in estimating realistic loads for a considerable submergence of the deck box has also been a part of the background for this recommendation. This represents an increase in airgap requirements of about 30% of the ULS crest height.

Another important difference between the two rule regimes is related to verification of the airgap predictions. For the maritime regulation, it is required that model tests should be used for verification if a designer go below the minimum recommendations put forward in Ref. [3]. In Norsok N-003, [4], however, it is explicitly recommended to verify the airgap analysis by high quality model tests. It should also be kept in mind that requirements put forward in Ref. [3] points to a ULS level analysis, while the recommendations of Ref. [4] apply to both ULS – and ALS – analyses.

As seen from the above paragraphs, there can be huge differences between recommended airgaps for RR1 and RR2, respectively. A proper assessment of the safety consequences of these differences for platform and crew is not known to this author.

Uncertainties are associated with the various assumptions regarding modelling metocean characteristics for airgap assessments. Some of these will be discussed in the following.

4 AIRGAP ASSESSMENT OF FIXED PLATFORMS

4.1 Still Water Level

For a fixed platform we will follow RR1. For airgap assessments, ULS still water level is mean still water level (MSL) + max tidal amplitude + 10^{-2}- annual probability storm surge, $SS_{0.01}$. In connection with ALS, still water level is MSL + 10^{-4}- annual probability storm surge, $SS_{0.0001}$. One can question why tide is not included at ALS level. However, a joint statistical analysis of crest height, tidal amplitude and storm surge, suggests that just adding the 10^{-4} – annual probability storm surge to the 10^{-4} –annual probability crest height, $c_{0.0001}$, gives a rather good approximation to the height of $c_{0.0001}$ above MSL. But this is valid for tide and storm surge variations experienced at the Norwegian Continental Shelf! It has no general validity.

4.2 Extreme Incoming Crest Height

<u>Design Wave Approach</u>

A possible estimate could be obtained by fitting a Stokes 5th order profile to the 10^{-4} annual probability wave height. Provided a proper associated wave period is chosen, this could give a reasonable conservative quasi-static global loads on the structure (e.g. foundation loads), but it may possibly be non-conservative for global failure mechanisms at higher levels of the structure which could be more sensitive to local loads caused by breaking waves.

The crest height of this Stokes 5th order wave is lower than the 10^{-4} – annual probability crest height, $c_{0.0001}$, for depths of – say - 60m or deeper. Therefore, the Stokes 5th order wave profile, defined in terms of wave height, is not adequate for determining the still-water airgap for design purposes.

Long Term Analysis of Crest Height

Since ULS and ALS crest heights are defined in terms of their annual exceedance probabilities, 10^{-2} and 10^{-4}, respectively, it seems reasonable to estimate these quantities using a stochastic long term analysis. Denoting the 3-hour maximum crest height by C_{3h} and assuming that a sea state is fully characterized by significant wave height, H_s and spectral peak period, T_p, the long term distribution of C_{3h} is given by:

$$F_{C_{3h}}(c) = \int_h \int_t F_{C_{3h}|H_sT_p}(c|h,t) f_{H_sT_p}(h,t)\, dt dh \qquad (1)$$

$F_{C_{3h}|H_sT_p}(c|h,t)$ is the conditional distribution of 3-hour crest height given the sea state and $f_{H_sT_p}(h,t)$ is the joint probability density function for H_s and T_p. As $F_{C_{3h}}(c)$ is known, the q-probability crest height, c_q, is found by solving:

$$F_{C_{3h}}(c_q) = \frac{q}{2920}\ , \qquad (2)$$

where 2920 is the no. of 3-hour windows per year. When using 2920 in Eq. (2), it is tacitly assumed that all 3-hour extreme crest heights are statistically independent. This is not fulfilled and Eq. (2) is assumed to result in slightly conservative ($\approx 5\%$) crest heights if all involved distributions are perfectly known.

For airgap assessments one should realize that the sea surface elevation should be modelled as a non-Gaussian stochastic field. A second order random process is frequently adopted in practice. The distribution function for global crest heights (largest crest height between zero-up-crossings), C, is given by, [5]:

$$F_C(c) = 1 - exp\left\{-\left(\frac{c}{\alpha h}\right)^{\beta}\right\} \qquad (3)$$

α and β are modelled as functions of mean sea state steepness, $s_1 = 2\pi h/(g t_1^2)$, and the Ursell number, $u = h/(k_1^2\, d^3)$. For long crested sea (2-dimensional sea), the functions read:

$$\alpha = 0.3536 + 0.2892 s_1 + 0.1060 u \qquad (4)$$
$$\beta = 2 - 2.1587 s_1 + 0.0968 u^2 \qquad (5)$$

where h is the significant wave height, t_1 is the mean wave period, k_1 is the wave number corresponding to t_1, g is acceleration of gravity and d denotes water depth. Expressions for α and β for short crested sea are also given in Ref. [5].

Assuming that global crest heights in a 3-hour stationary sea state are statistically independent (not perfectly fulfilled) and identically distributed, the distribution function for the 3-hour maximum crest height is given by:

$$F_{C_{3h}|H_sT_p}(c|h,t) = \left[1 - exp\left\{-\left(\frac{c}{\alpha h}\right)^{\beta}\right\}\right]^{n_{3h}(t)} \qquad (6)$$

A second order random model does not account for wave breaking. A consequence of this is that the predictions of crest height for very steep sea can be conservative. For less steep sea states (but still reasonably steep sea), second order prediction may possibly be slightly non-conservative regarding the most severe crest heights due to higher order effects. Regarding airgap assessments, second order crest predictions should be considered as being slightly on the low side (see Table 3) unless the predictions are properly verified for the case under consideration. This will be further discussed later.

Example of a Long Term Analysis of Crest Height

A joint distribution of H_s and T_p at a southern location on the Norwegian Continental shelf is presented in Ref. [6]. The estimation is based on NORA10 hindcast data, [7], from 56°11'N and 3°41'E. The data used in this study represents data for every 3 hours from September 1957 through June 2014.

The joint density function of H_s and T_p, $f_{H_sT_p}(h,t)$, is written as a product of a marginal density function for H_s and a conditional density function for T_p given H_s. A LoNoWe distribution is used for H_s. This is a hybrid model with log-normal distribution for values of H_s below η and a 2-parameter Weibull distribution for $h > \eta$, [8]:

$$F_{H_s}(h) = \begin{cases} \Phi\left[\dfrac{\ln(h) - \mu_{\ln(H_s)}}{\sigma_{\ln(H_s)}}\right]; & h \le \eta \\ 1 - exp\left\{-\left(\dfrac{h}{\alpha_{w2}}\right)^{\beta_{w2}}\right\}; & h > \eta \end{cases} \qquad (7)$$

For $h \le \eta$ it is utilized that if H_s is log-normal distributed, the variable $ln(H_s)$ is normal distributed with parameters $\mu_{\ln(H_s)} = E[\ln(H_s)]$ and $\sigma^2_{\ln(H_s)} = VAR[\ln(H_s)]$. The following parameters were estimated in Ref. [6]:

$$\mu_{\ln(H_s)} = 0.497, \sigma_{\ln(H_s)} = 0.6463, \eta = 2.8m, \alpha_{w2} = 1.992\ and\ \beta_{w2} = 1.351\ .$$

Continuity is required both for distribution function and density function at $h = \eta$, i.e. the LoNoWe model is actually a 3-parameter model. An alternative model, more frequently used, is a 3-parameter Weibull model, but in Ref. [6] the LoNoWe model is used.

A log-normal distribution is used for the conditional distribution of T_p given H_s:

$$F_{T_p|H_s}(t|h) = \Phi\left[\frac{\ln(t) - \mu_{lnT_p}(h)}{\sigma_{lnT_p}(h)}\ |\ h\right]; t > 0 \qquad (8)$$

For each class for H_s of width 1m the distribution parameters are estimated. In order to obtain estimates for the parameters beyond the level of observations, smooth functions are fitted to the point estimates. Thus the parameters actually used for long term analyses are given by, [6]:

$$\mu_{lnT_p}(h) = 1.7392 + 0.2291 h^{0.5965}$$
$$\sigma^2_{lnT_p}(h) = 0.0055 + 0.0913 exp\{-0.5716 h\} \qquad (9)$$

The fitted distribution for H_s is compared to the empirical distribution in Figure 3 and a reasonable good fit is seen for the interesting sea severity, i.e. stormy seas. A conditional 90% band for T_p given H_s is shown versus H_s in Figure 4, where the hindcast data also is presented. Estimated ULS and ALS characteristics are given in Table 1.

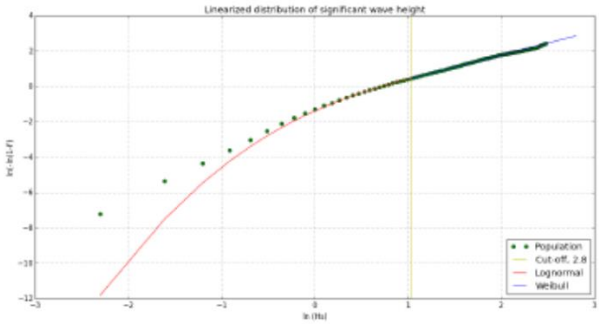

Figure 3. Fitted LoNoWe distribution for H_s versus empirical distribution. From Ref. [6].

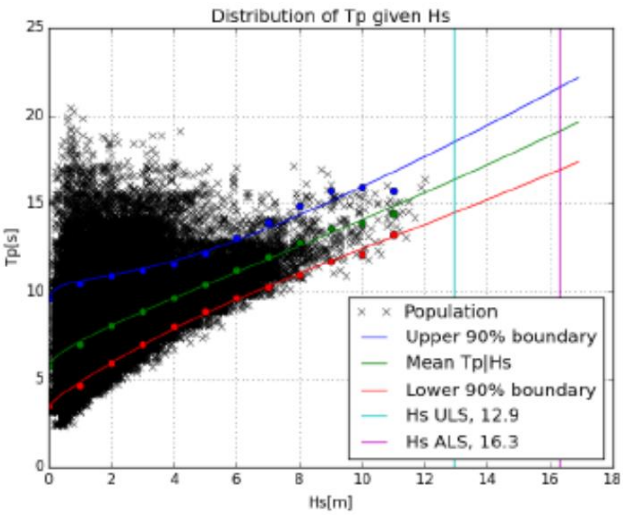

Figure 4. Estimated conditional 90% range for T_p given H_s. From Ref. [6].

Table 1. 10^{-2} - and 10^{-4} – annual probability significant wave height and associated spectral peak period. From Ref. [6].

Design case	H_s (m)	$\overline{T}_p$ (s)
ULS	12.9	16.4
ALS	16.3	19.2

The long term analysis for crest height uses the joint function described above for the weather characteristics and the Forristall crest height model, Eqs. (3) - (5), [6]. A separate long term analysis of wave height (trough to crest height) of global waves is also performed using Forristall wave height distribution, [9]. The ULS - and ALS crest heights and wave heights are given in Table 2. It should be noted that the 10^{-4} annual probability crest height (ALS) is about 30% larger than the 10^{-2} – annual probability crest height (ULS). If we fit a Stokes 5th order wave profile to the ALS wave height and the conditional mean wave period, the crest height of the Stokes 5th order profile will typically be around 58% of the wave height. Applying this ratio to the ALS wave height in Table 2, the "ALS crest height" is found to be 18.4m, i.e. about 1.2m lower than the 10^{-4} annual probability crest height. This is just an indication. For other depths and steepness values, the underestimation may differ slightly from this value.

Table 2. 10^{-2} - and 10^{-4} – annual probability crest height, C, and wave height, H. From Ref. [6].

Design case	H (m)	C (m)
ULS	24.6	15.1
ALS	31.7	19.6

4.3 Uncertainties Associated with Incoming Crest Height

Long Crested or Short Crested Sea

Regarding prediction of crest extremes in a point, Figure 5 and Figure 6 suggest that there is a rather small difference between long crested and short crested predictions within a second order assumption for the sea surface. However, the area or volume of platform topside being submerged in case an extreme wave crest exceeds the available airgap, the difference between long crested sea and short crested sea may be more clearly pronounced. From model tests, impact loads due to loss of airgap seem to be smaller in a short crested case. How much smaller is the crucial question? This will to a large extent be governed by the shape of the worst wave event. If the critical wave is of a pyramidal shape, difference will be very large, while if the crest length is 50-100m, i.e. comparable to deck dimensions, the effect may not be that significant.

For a further investigation of short crestedness and its effect of the shape, of the most extreme steep crest heights possibly in a development towards wave breaking, a fully non-linear model of the surface process or model test experiments should be considered, see Ref. [16] where it is indicated that crest length seems to increase with increasing crest height in spread sea. In case the extreme wave is not in a development towards breaking, its shape is indicated by a second order new wave profile i.e. the Fourier transform of the underlying wave spectrum modified to account for second order effects.

Point Maximum Versus Area Maximum

In a random wave field, a particular crest will evolve in time and space. A consequence of this is that even if an extreme crest event does not reach the double bottom (bottom of deck steel) at the edge of the deck, it may grow to cause a major wave-deck impact under the deck. To avoid such an event with sufficient margin, target crest height should be the area maximum and not the maximum in a given point.

The difference between an area maximum and a point maximum is discussed in Ref. [11]. The area investigated is

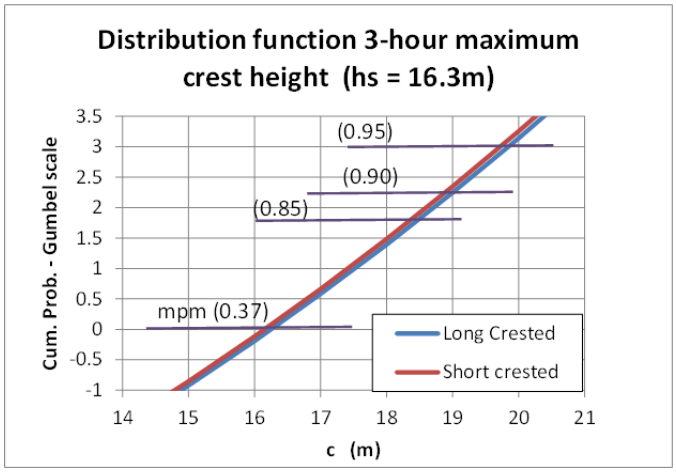

Figure 5. 3-hour extreme value distribution, Eq. (6), for the peak sea state along the ULS contour.

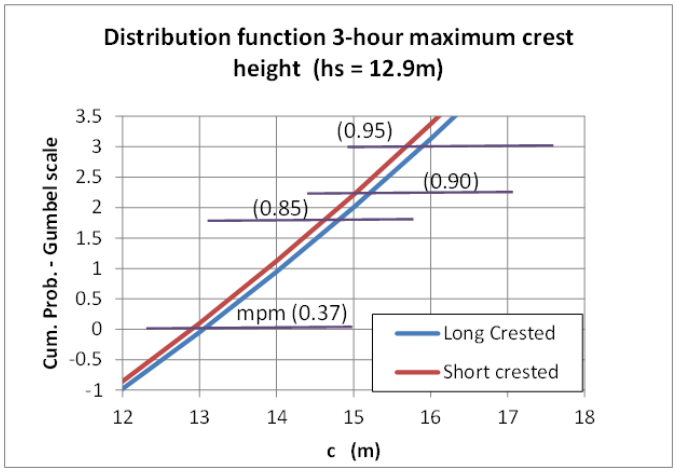

Figure 6. 3-hour extreme value distribution, Eq. (6), for the peak sea state along the ALS contour.

quadratic with side length denoted L. Wave spectrum is a JONSWAP spectrum with peakedness parameter of 2.5. Short crested sea is adopted and is characterized by a simplified version of Ewans spreading, [19], see Ref. [11] for details. It is proposed that the difference between the expected areal maximum and a point maxim is a function of L/λ_1, where λ_1 is the wave length corresponding to the mean wave frequency, m_1/m_0. m_j is wave spectral moment of order j. Based on a quadratic fit to simulated results, the difference is estimated by, [11]:

$$\frac{E[Cmax|\ area]-E[Cmax|point]}{\sigma} = 0.9829 + 0.4170\ln\left(\frac{L}{\lambda_1}\right) + 0.0427ln^2\left(\frac{L}{\lambda_1}\right) \quad (10)$$

σ is the standard deviation of the sea surface process and equal to 25% of the significant wave height of the sea state.

As an example, let us look at the ALS case of Table 1. The wave length is here assumed to about 360m for a depth around 100m. Assuming deck side is 60m, the increase of maximum crest height due to area effect is about 1.5m according to Eq. (10), i.e. slightly less than 10% increase. This is for a JONSWAP spectrum with spectral peak value, γ, of 2.5 and the approximate Ewans spreading.

Adequacy of Second Order Assumption Regarding Estimation of Extreme Crests

The governing contribution to the required airgap is the height of the ULS wave crest and the height of the ALS wave crest obtained using a second order model for the surface process. An important question to address is the adequacy of the second order model. At some steepness level, higher order effects will be of importance. Initially, higher order effects will result in a slight increase of the crest percentiles. However, as the local steepness reach a limiting value, wave breaking is introduced and the highest crest height percentiles will be reduced. Since wave breaking will not take place in a second order sea, we can, for extreme cases, see that the true percentile values become lower than the second order percentiles. For a slightly longer spectral peak period, the limiting effect of wave breaking is shifted to a lower exceedance probability making the latter sea state to the critical sea state regarding airgap assessments.

During the last decade several authors have compared second order theory with model test data and full scale data, see e.g. [12], [13], [14] and [15]. The adequacy of the second order assumption for estimating extreme wave crest heights is investigated using full scale measurements from the Northern North Sea in Ref. [14]. Water depth is 190m and 20-minute time series of surface elevation are estimated using a SAAB radar with a sampling frequency of 7.68Hz. Time series are in principle available for all 20-minute windows with significant wave height higher than 5m. Of course there are gaps in the data, but a considerable amount of apparently good quality data is available.

Data series are pooled into classes in terms of significant wave height and spectral peak period. Class widths are 1m and 1s, respectively. Results in terms of normalized crest height are shown in Figures 7-9 for 3 sea state classes. The empirical exceedance distributions for crest heights based on the 20-minute time series are shown in red. For each data set, the crest heights from the individual 20-minute series are merged and the empirical distribution based on the merged samples is shown in green. The Forristall crest height distribution for short crested sea is shown in blue.

For the 20-minute series, quite some scatter around the second order curve is seen. This should be expected since the number of global crests (largest crest between zero-up-crossings) during the selected 20-minute time series is in the range 100-140. For the merged crest height samples, the bulk of the empirical distribution represent a rather close fit to the second order crest height distribution. The latter is calculated from Eq. (3) using the average values of H_s and T_p for the data in each class.

For Figure 7, an accurate fit is obtained for exceedance probabilities well below 10^{-3}. The cumulated duration of the data shown in Figure 7 is 375hours. A higher sea state is considered in Figure 8. Fewer observations are available and

the cumulative duration is 57 hours, i.e. 6-7 times less crests than for Figure 7.

Due to fewer data points, the exceedance probability of the largest crest height of the class in Figure 8 increases with almost an order of magnitude as compared to Figure 7. A similar increase in the probability level for good agreement with the second order distribution is seen. This indicates that as more observations become available, the empirical distribution seems to approach the second order model. But, for the first case, the empirical upper tail suggests extreme crest heights slightly less than what second order suggests, while for the other case, which is associated with smaller sea state steepness, crest heights higher than second order predictions are suggested for low exceedance probabilities. Is this observation possibly indicating a steepness dependence of the adequacy of the second order model?

It would be convenient if we can conclude that the second order distributions suggested by Forristall, [5], is of good accuracy for the range of concern, and that the variation we see regarding the upper tail can be considered as statistical noise. This would be in agreement with the conclusions in [15]. However, the case shown in Figure 7 has a higher sea state steepness than the case shown in Figure 8. This could also suggest that the reason for upper tail below second order in Figure 7 are due to wave breaking. The less steep sea state in Figure 8 is not limited by breaking to the same extent and crest heights above second order are observed.

The argument of wave breaking as a possible explanation for the deviation seen in Figure 7, can, however, be questioned by the results shown in Figure 9. This is based on data from one 24-hour storm period where the significant wave height remained between 9.6m and 11.9m. The sea state steepness of the case in Figure 9 is similar to the steepness of the case in Figure 7, but there is no pronounced effect of wave breaking in Figure 9. For this case, the empirical distribution gives a systematic exceedance of the second order crest height percentiles for exceedance probabilities below 10^{-2} for an arbitrary crest height.

Of course it is not sea state steepness that defines breaking, one should rather look at the steepness of the largest wave groups in the sea states. The local spread of a sea state i.e. the short crestedness for the largest wave groups could well be an interesting parameter regarding development large crests. This is a quantity that is not well known and further investigation on the variation of short crestedness of extreme storms is recommended. An attempt to discuss this is found in [16].

Results for a 3-hour period with significant wave height around 12m and a spectral peak period of 15s, i.e. $s_p = 0.0342$, are also shown in [14]. The empirical distribution fits very well to second order distribution, except for the upper 5 crests which lie slightly above the corresponding second order percentiles.

A reasonable conclusion in view of present state of knowledge is to consider crest height results obtained using a second order assumption as a lower bound for wave crest height. The underestimation can in realistically spread sea be about 5-10%.

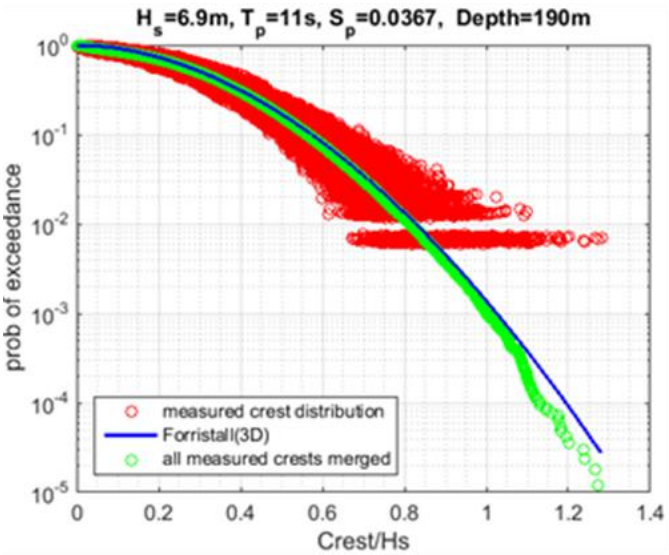

Figure 7. Crest height exceedance probabilities compared to second order, 375 hours of measurements with $H_s \in [6.51m, 7.50m]$ & $T_p \in [10.51s, 11.5s]$. From Ref. [14].

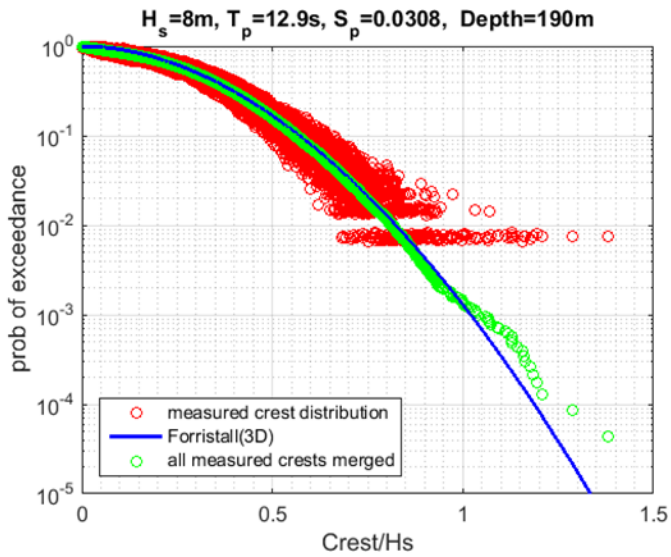

Figure 8. Crest height exceedance probabilities compared to second order, 57 hours of measurements with $H_s \in [7.51m, 8.50m]$ & $T_p \in [12.51s, 13.5s]$. From Ref. [14].

Uncertainty in High Frequency Decay of Wave Spectrum

The parameterized model for second order global crests, Eqs. (3) – (5), was established using the JONSWAP spectrum. The high frequency tail of this spectrum decays proportional to f^{-5}. In 1984 Phillips, [22], suggested that the high frequency tail should rather decay proportional to f^{-4}. Will this affect the distribution of global crest heights? Two models for the wave spectrum was compared in a small model tank at Marintek (Lilletanken) as a part of a master thesis, [17]. The input sea state was rather steep and given by, H_s = 10m & T_p

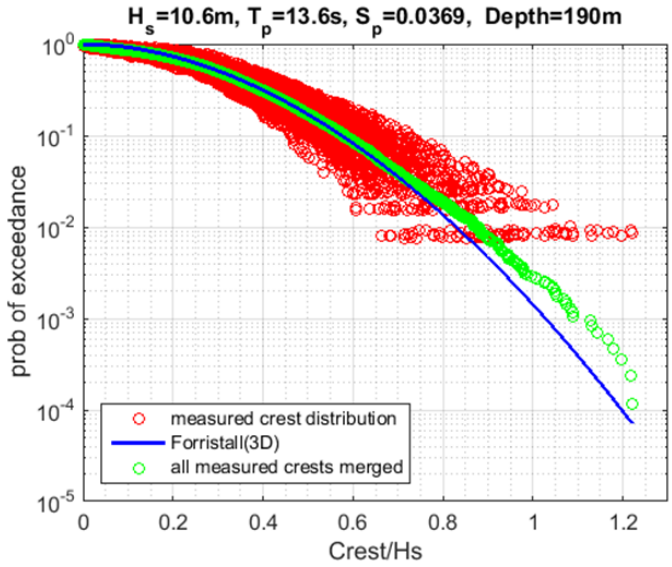

Figure 9. Crest height exceedance probabilities compared to second order, 24 hours of measurements during January 26, 2012. $H_s \in [9.6m, 11.9m]$ & $T_p \in [13.3s, 14s]$. From Ref. [14].

= 10.55s. For the JONSWAP spectrum decaying according to ω^{-5}, the average significant wave height in the target position in the tank after ten repeats (with random seed) of 1-hour (full scale duration) was estimated to 9.31m. Using the same procedure with the Donelan spectrum which is a version of the JONSWAP spectrum decaying according to ω^{-4}, the average significant wave height was 9.17m. Estimated crest height exceedance probabilities for the two spectral models are shown in Figure 10. The figure is based on pooling global crest heights from ten 1-hour (full scale time) model test series.

It is clear from Figure 10 that the spectral decay is not very important regarding crest height for the undisturbed crest height distribution. However, merely one sea state is considered so further work should be done before a robust conclusion can be made. The resulting shape of the basin wave spectrum is also associated with uncertainties. It is recommended to investigate the high frequency sensitivity using a second order simulations of the surface process for two spectral models.

Effects of Current

Usually current is not considered important for airgap assessments of fixed platforms. Current will affect a given wave field but it is presently not known whether it will improve or worsen the airgap scenario. Here we will neglect the effect of current.

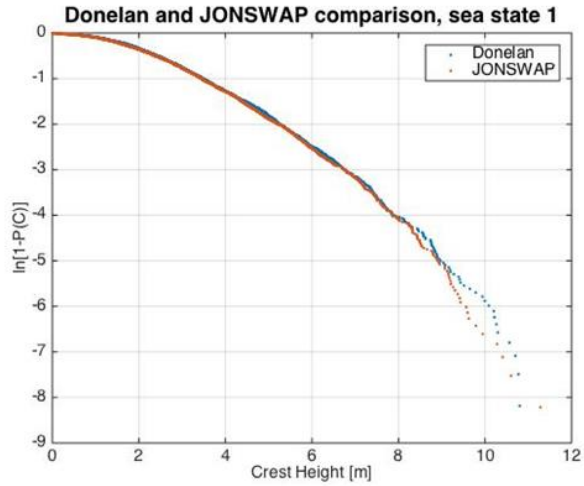

Figure 10. Comparison of exceedance probabilities for crest height based on ten 1-hour simulations from JONSWAP – and Donelan spectra, respectively. From Ref. [17].

4.4 Wave – Structure Interaction

Jackets and Jack-ups

Both jackets and jack-ups are in most cases assumed to be sufficiently transparent to neglect diffraction/reflection from the platform. This should be fine for a typical jack-up, but should be assessed on a case by case for jackets.

Gravity Based Platforms

For gravity based platforms one needs to account for wave – structure effects when determining required airgap. Depending on water depth and height of caisson, a considerable diffraction/reflection effect of the amplification of the incoming wave crests must be expected. The diffraction effect of a regular wave with 19.3m crest height and a wave period of 15s is indicated in Figure 11. The maximum crest height under platform deck varies from 18 – 26m. For an irregular sea state of similar severity and steepness, the surface field under the platform deck would be much more chaotic.

As waves are approaching a gravity based structure, the disturbed crest heights will typically be considerably higher than the undisturbed incoming crest height. This is illustrated in Figure 12 showing model test results for the crest amplification factor of regular waves with various heights and directions. The results for the 40m wave height should not be given too much attention. For this wave, the crest height hit the deck building of the model and the large values are a result of water flowing up along the front. Furthermore, the 40m wave height is also 15% larger than the 10^{-4} – annual probability height for the platform area.

It is seen from Figure 11 that amplification is varying in space. It is also dependent of wave height, wave period and wave direction. But in spite of this, Figure 12 suggests that an amplification of 15 – 40% could be expected for this type of platform. Except for the region close to columns, a linear

diffraction analysis may possibly be sufficiently accurate, but the rather chaotic pattern of the surface between the platform legs does suggest a verification by a proper model test experiment if airgap seems to be a critical topic. Close to the columns non-linear diffraction and run-up must be expected and deck structure close to the columns must be designed to take impact loads from run-up events.

The amplification indicated by Figure 12 is expected to be representative for long crested seas at the same depth. For less depths, a much larger amplification should be expected, see e.g. [23] and [24].

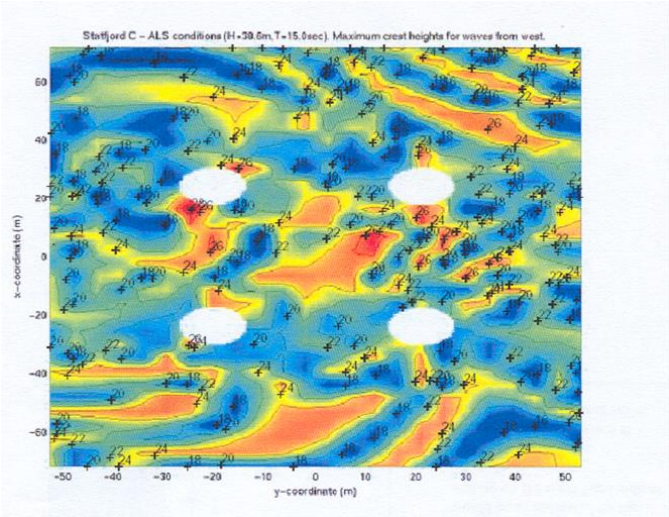

Figure 11. Maximum disturbed crest (upwell) under a gravity based structure for a harmonic wave with amplitude 19.3m and wave period 15s. (Figure prepared by Per Teigen, Statoil.)

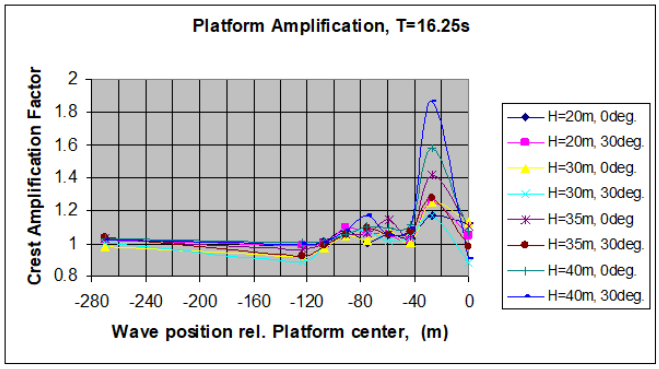

Figure 12 Amplification of crest height for a regular wave approaching a gravity based structure in a water depth of about 150m.

4.5 Concluding Remarks for Fixed Platforms

As an illustration of the expected contributing quantities to the required airgap, Table 3 is prepared. The crest height data are for the position represented by the hindcast data. The other numbers are typical numbers from the author's point of view based on discussions in previous sub-chapters. It is clearly seen that it is the incoming crest height and its associated uncertainties that is of major concern.

It is important to note the difference between the ULS and ALS airgap. The deck should at least be higher than ULS airgap. But it shall also withstand loads associated with the ALS wave event without escalation. The worsening from ULS to ALS indicated in Table 3 is a possible submergence of topside of 4-6m depending on type of structure. The complexity in determining impact loads for such a level of submergence and the associated uncertainties, suggests that best solution is to select an airgap close to the ALS airgap.

Table 3. An illustrative calculation of required airgap for fixed platforms.

Contribution to required airgap	Jack-ups & Jackets		Gravity based structures	
	ULS	ALS	ULS	ALS
Tidal amp. (m)	1.0	-	1.0	-
Storm surge (m)	0.9	1.1	0.9	1.1
Second order crest height (m)	15.1	19.6	15.1	19.6
Uncertainty 2. order (m)	0.8 +/- 0.8	1.0 +/- 1	0.8 +/- 0.8	1 +/- 1
Area maximum (m)	1.5	2.0	1.5	2.0
Platform-Wave Interaction	-	-	3 +/- 1.5	4 +/- 2.0
Target airgap (m)	**18.5 – 20.1**	**22.7–24.7**	**20.0 –24.5**	**24.7 – 30.7**

5 AIRGAP ASSESSMENT OF FLOATING STRUCTURES

5.1 Still Water Level and Draft of Floater

For a floater the varying level of still water surface is of no concern. What is of concern is the draft of the floater.

Drilling semi-submersibles will typically follow the Maritime Regulation, RR2 (See Chapter on Rule Regimes). At the drilling location, the platform will under normal condition stay at an operational draft, while in case of severe weather forecast, the rig is de-ballasted to the survival draft. The critical still water airgap is the still water airgap in the survival condition. There is a considerable difference between various drilling rigs concerning still water airgap in survival condition. For 7 drilling rigs of semi-submersible type, the still water airgap in survival condition varies from 13.5m to 18.5m, [20]. This is a considerable difference – in particular since the required airgap is established based on the same regulation regime, RR2.

A Statoil operated semi-submersible, Veslefrikk B, was installed in the Northern North Sea in 1989. The platform was originally build as a drilling rig, but was modified to a production rig in agreement with 1989 version of RR1. The platform was operated on one draft with a still water airgap of 14.5m. In mid-nineties, a major wave – deck impact was experienced. A new model test program was carried out for a set of 10^{-2} annual probability sea states. Due to the selected scale, ALS sea states could not be tested. The result of this test program was to introduce a survival draft resulting in a still water airgap of 17.5m. It is therefore somewhat surprising

that newer rigs have a survival still water airgap considerably less than this figure.

Semi-submersibles used as floating production units will be designed in agreement with PSA rule regime, RR1. These will typically operate at one draft irrespective of weather conditions. Still water airgap is in most cases around 20m.

5.2 Method of Analysis – Long Term or Short Term Approach

No matter of adopted regulation, the required minimum airgap is defined in terms of a maximum permissible annual exceedance probability, q. Both rule regimes require that for an annual exceedance probability of 10^{-2}, a wave crest should not hit areas where crew could be under such weather conditions. In contrast to fixed platforms, the critical crest height will not necessarily be the 10^{-2} - annual probability undisturbed crest height at platform site. For floaters one also needs to account for platform motions and, additionally, one must account for wave- structure interactions.

Regarding platforms operating in agreement with RR2, there is no requirement regarding model test. Motion and airgap assessments are in most cases done by frequency domain methods where it is tacitly assumed that the sea surface elevation can be modelled as a stationary and homogeneous Gaussian random field. This may be of sufficient accuracy for motions, but regarding airgap it is too crude. This has been recognized for a long time and the incoming crest height is corrected by multiplying Rayleigh extremes with an asymmetry factor. The asymmetry factor varies under the platform. Reference [3] is discussing this variation and they recommend that a maximum value of asymmetry factor is recommended to be 1.3 at the up-wave deck front. In case the governing sea state is very steep, the amplification factor of 1.3 may possibly be conservative due to wave breaking. Accounting for this, the governing sea state regarding airgap assessments may be shifted to a less steep sea state along the contour and the factor 1.3 may still be valid. One should not utilize a lower asymmetry factor than the values recommended by Ref. [3] without verifying the selected asymmetry factor by a properly designed model test experiment.

For structures following RR1 (PSA regime), it is also required that one shall also validate a sufficient still water airgap in case platform is hit by the 10^{-4} – annual probability disturbed wave crest (ALS case) accounting both for platform motions and sea-structure interactions. There is no requirement for no wave-deck impacts in the ALS case, but it is required that the platform is designed against possible ALS impacts. It is standard practice for production semi-submersibles on the Norwegian Continental Shelf to validate proposed still water airgap with thorough model test experiments. The model test surface is assumed to represent a proper deviation from the Gaussian random field and there is no need to correct with the abovementioned asymmetry factor.

In order to assess required airgaps for floaters at a given annual exceedance probability, a full long term analysis is in principle required. This will be more or less impossible in practice if we are to account for the non-Gaussian structure of the surface field, a simultaneous modelling of weather characteristics that could affect platform heave, pitch and roll (wind sea, swell sea, wind speed) and sea - structure interaction. A simplified approach is to select a set of short term metocean conditions based on q – annual probability metocean contour lines for $q = 10^{-2}$ and $q = 10^{-4}$ for ULS and ALS, respectively, and use these as design conditions. Using the methods outlined for metocean contour method, see e.g. Ref. [10], long term extremes can be estimated using short term analyses. In particular, this approach is convenient if model tests are required in order to describe the short term variability properly.

5.3 Metocean Contour Application

<u>Method description</u>

Examples of ULS and ALS metocean contour lines are shown in Figure 13. In principle, one could also include mean wind speed as a third weather characteristic. This could be convenient if wind induced rolling is of importance for the estimation of extremes for the relative wave crest height. In this paper, however, we will consider contours for significant wave height and spectral peak period and briefly review the steps involved in using the method.

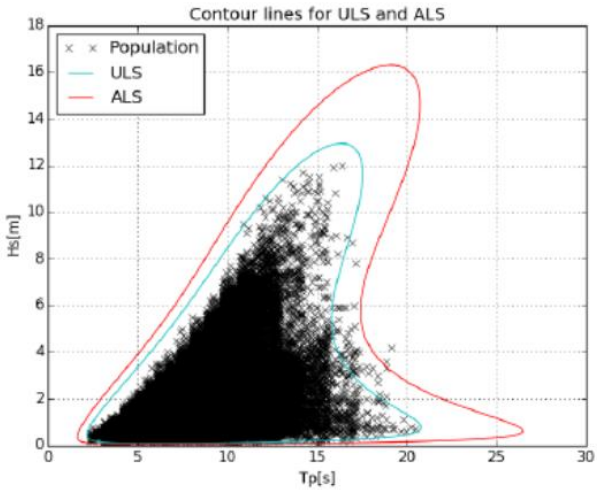

Figure 13. Metocean contour lines. From Ref. [6].

As the contours are available, the first step is to determine the critical sea state for airgap assessments along the ULS - and ALS-contour, respectively. This is more challenging for a floater than for fixed platforms since airgap is very much effected by platform motions and wave-structure interactions. A possibility is to select a range on the q-probability contour line which is enveloping the critical sea state for airgap assessment. Selecting 5-6 sea states to represent the range, one could do a screening study to identify the worst sea state within the range.

As the worst sea state is identified, the aim is to establish the distribution function for the 3-hour extreme relative crest height – either by numerical analyses or - preferably - from model tests. Irrespective of method of analysis, at least 20-30 repeats of the critical sea state should be done. The q-probability relative crest height can finally be estimated by the

α-fractile of the 3-hour extreme value distribution. For ULS, α = 0.9 is a reasonable (slightly conservative) choice, while α = 0.95 is expected to be better for ALS. The metocean contour method is an approximate method. The approximation lies in the fact that the equivalent fractile level for obtaining a good estimate for the long term extremes by a short term variable is dependent on both the response properties and how contours are determined.

The underlying "true" percentile depends on the coefficient of variation (CoV) of the 3-hour maximum response. It is the relative importance of this source of variability compared to the long term variability of the slowly varying weather characteristics that is likely to affect the target percentile. Experience suggests that as far as CoV is less than 0.2- 0.25, the percentiles for ULS and ALS suggested above is reasonable or slightly conservative. For airgap assessments the target variable is 3-hour maximum relative crest height. For this variable, CoV is less than 0.2 and the percentiles recommended above should be reasonable approximations.

If, however, focus is on impact loads from breaking waves, the COV could well be 0.5 – 1.0. In such a case, the underlying true percentiles would be much larger! A percentile of about 0.999 is not unrealistic. The reason for the large percentiles is related to the fact that the projection of the design point (most likely combination of variables as the q-probability is realized) into the H_s-T_p plane is far away from the q-probability contour of H_s and T_p. The percentile of the q-probability impact in the design point is a very high say in the order of 0.9999. Our lack of understanding of the short term variability of 3-maximum impact pressure suggests that one should be very careful to put too much weight to percentile well beyond 0.99. For the breaking wave impact problem, one should be very careful by applying both long term analyses and metocean contour line approaches. Better understanding of the of the short term variability of impact loads from breaking waves are strongly needed!

In contrast to the q-probability impact load due to breaking waves, the q-probability crest height problem can be investigated both by long term assessments and metocean contour approaches. No matter of method, predicting the q-probability **relative** crest height for a floater will be associated with larger uncertainties than for the q-probability disturbed crest height for a fixed platform. Various sources of uncertainties will be discussed below.

5.4 Sources of Uncertainties

Selection of Design Sea States

It is more complicated to find the worst sea state along the q-probability contour for a floater than for fixed platform. The best approach is possibly to do a proper screening along a selected range of the target contour. A robust screening study would require several repeated tests for the screening sea states since we primarily are interested in the extremes of the 3-hour maximum relative crest height. The critical point under the platform projection will vary somewhat from repeat to repeat of the given sea state due to short term variability of the maximum relative crest height under platform deck. The critical point will also depend on characteristic period (preferably spectral peak period), mean direction of propagation of waves, significant wave height and the degree of short crestedness of the sea state. Wind and current may also be of importance for indicating the criticality of a sea state. Wind because it may affect slow drift motions of floater, while current can be of some importance if it affects the wave –structure interaction, [18]. It is also worthwhile to mention that presence of swell may be of importance depending of the sea states selected for the screening study, i.e. choice of wave spectrum may be of importance. Finally, if the floater is operating using dynamic positioning to stay close to target position, the resulting effect of varying wind and corresponding varying total thrust on platform motions, e.g. platform rolling, must be captured.

In practice one has to simplify the screening process. But with increasing simplification, more effort is necessary to verify that the final recommendation is on the safe side regarding required still water airgap and loads from possible impacts. One of the most important advantage of including the ALS requirement in RR1 regarding airgap assessment is that it ensures a margin well above the 10^{-2} – annual probability still water airgap. Of course, if the ALS weather scenario is too simplistic, the numerical analyses or model test experiments will not necessarily result in a robust estimate of ALS airgap. But the ALS check will at least ensure a robustness well beyond the ULS level for events not captured by the prescribed ULS scenario.

Long Crested or Short Crested Sea

The real ocean surface for wind induced sea is short crested. A thorough discussion of previous models for wave spreading is presented in [19] in connection with a careful assessment of spreading data from Maui. It is clear from the paper that, based on the Maui data, the standard deviation of the spreading is smallest around the spectral peak and that it broadens for both lower frequencies and higher frequencies. For high frequencies, the spreading function seems to attain a double peaked shape. For fixed platforms, utilizing long crested sea will yield conservative results. This is not case for a semi-submersible. Short crested sea may result in platform rolling even in head sea.

Here we will focus on the spreading around the spectral peak, i.e. around the most energetic frequency band. From the Maui data, standard deviation of the spreading around the peak frequency vary from 15° to 30°. A commonly adopted spreading function is:

$$d(\theta) = c(n)cos^n\theta; \; -90^o \le \theta \le 90^o \, , \qquad (11)$$

where

$$c(n) = \frac{\Gamma(1+\frac{n}{2})}{\sqrt{\pi}\,\Gamma(\frac{1}{2}+\frac{n}{2})}$$

Standard deviation of spreading in θ using Eq. (11) is shown in Figure 14. Regarding spreading around the peak frequency, n=14 seems to be conservative for cases where spreading is reducing the variable of concern, while n = 2 will be conservative for cases where spreading does worsen the situation (e.g. ship rolling in head sea). In Ref. [4], n=10 is

recommended as a lower limit for cases where spreading reduces the response. The average standard deviation from the Maui data suggest n=6 as a proper average for the range around the spectral peak frequency. If spreading is important for the variable under consideration, it will typically be non-conservative to use the mean. Thus n=4 may be reasonable for cases where spreading does worsen the situation, while n = 10 can be reasonable for cases where spreading reduces the response.

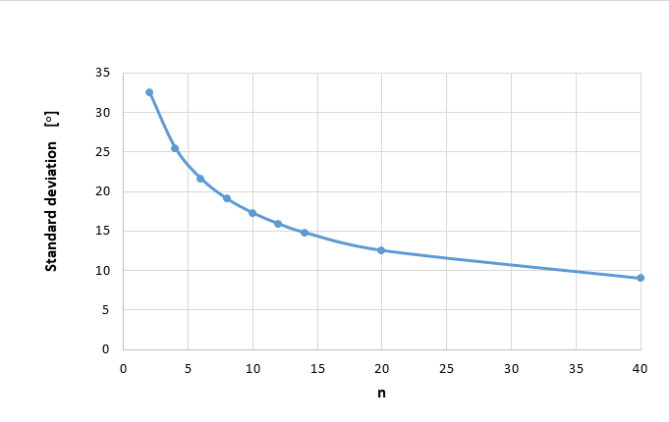

Figure 14. Standard deviation of spreading function of Eq. (11).

The spreading is often assessed based on sea states of moderate severity. Is there any reason to be concerned by the fact that design sea sates will have significant wave height of 15 – 20m, while data used for assessing spreading typically correspond to sea states less than 5m? Spreading is, furthermore, estimated based on averaging over the length of the data series. In case there is an underlying modulation of the spreading within a sea state, this is smoothed out by the averaging process. Could such modulation be of concern? Nature have never claimed to be neither stationary nor homogeneous, it is us that has imposed these restrictions. The spreading of concern for design is often the spreading of the largest individual waves or worst wave groups, in particular for fixed platforms, but also for wave-deck impact problems for floaters. This will be the scenario causing the worst response for ULS and ALS assessments. Is the spreading for these events equal to the average spreading of the full sea state? According to Ref. [16] there is a tendency of reduced spreading with increased crest height.

The spreading discussed above is spreading of a pure wind generated system. In practice, a sea state may well be of a combined nature, i.e. the local wind sea is superimposed on an incoming swell system which typically have a different direction of propagation. For fixed platforms, this is not expected to be important, but for floaters it may be of some concern. A swell system with a direction of propagation different from that of the wind sea, may cause unfavourable platform motions being uncorrelated with the steep wind induced extreme waves. It is not common to consider simultaneously occurring wind sea and swell from different directions, but may be this should be investigated if sea states along the steep side of the metocean contour are important. If this is done, short crested sea with n as the most unfavourable value between 4 and 10 should be utilized for the wind sea in order to investigate the effect of short crested sea on the airgap problem.

Choice of Wave Spectrum

No matter of response analysis, the spectral density function for the wave field is needed. The JONSWAP spectrum is frequently used for growing wind sea. For steep sea states with a moderate significant wave height, say Hs < 12m, the sea state can be of a combined nature. This means that the wind sea is superimposed on swell sea, which generally will come from a different direction. A spectral model that can handle the combined nature of the frequency spectrum is the Torsethaugen wave spectrum, [21], but a deviation in direction between the two systems is not build into the Torsethaugen model. A proper directional deviation between wind sea and swell sea must be introduced by the analyst. For some few sea states along the ULS contour line shown in Figure 13, the corresponding JONSWAP - and Torsethaugen spectra are compared in Figure 15. For the two lowest sea states, we see a certain trace of long period swell energy. For a floater, the Torsethaugen spectrum may well give worse results for these sea states. The long period swell may under unfavourable conditions result in heave - and roll contributions that would be independent the wind sea. This effect is missed when adopting JONSWAP.

The Torsethaugen spectrum is a sum of two JONSWAP like spectra. The major difference is that for high frequencies, JONSWAP decays proportional to f^{-5}, while Torsethaugen is based on the recommendation of Ref. [22] and decays proportional to f^{-4}. The Donelan spectrum is a version of JONSWAP spectrum decaying with f^{-4}. A comparison of crest height distributions from model tests using both these spectral models are shown in Figure 10. The model test results indicate that the effect on the crest height distribution is rather small – at least for cases investigated in Ref. [17].

As seen from Figure 15, that for sea states with spectral peak period of 14s and 16s, the JONSWAP and Torsethaugen wave spectra are more or less coinciding. Torsethaugen spectrum will decay slower and will have somewhat more high frequency energy, but this difference is not likely to be important compared to other uncertainties. It is interesting to note the difference between the spectra for a sea state to the right of the mode of the contour line. For the wave period band from 6-12s, the Torsethaugen model have more energy than the JONSWAP model. This can be important for airgap assessments of semi-submersibles.

For an airgap assessment of a floater, one should include sea states within the spectral peak period range form 10s to about 2s higher than the spectral peak period at the mode of the contour in the screening assessment.

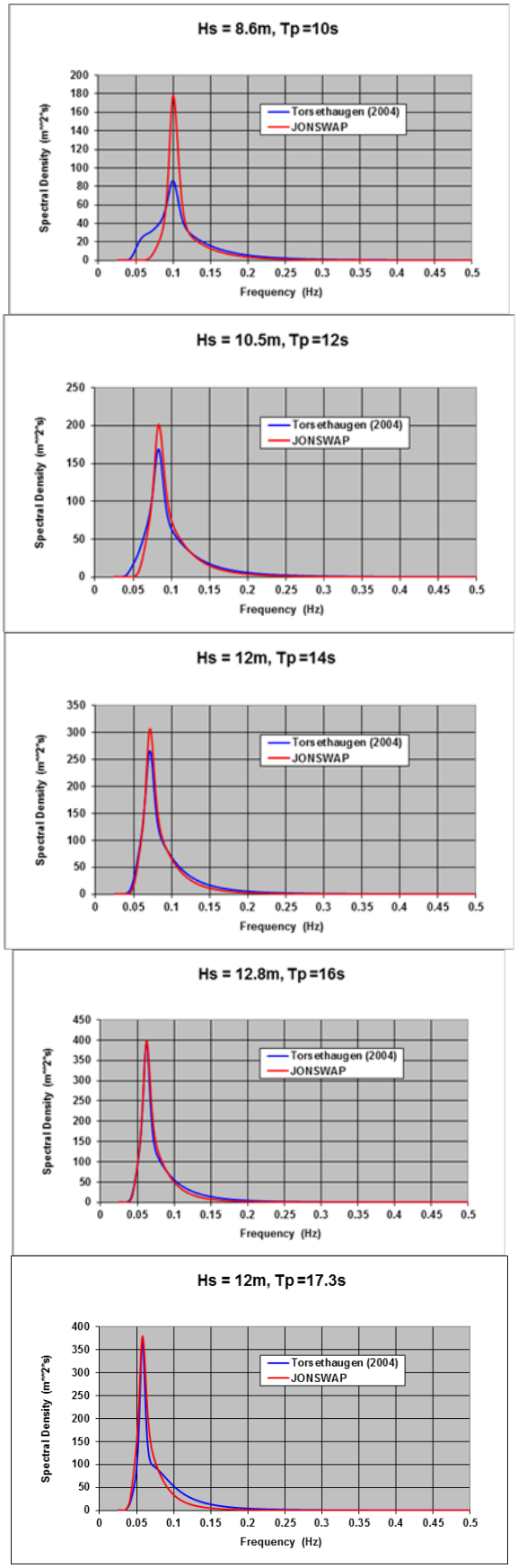

Figure 15 Comparison of JONSWAP – and Torsethaugen spectra along the ULS contour lines.

Disturbed Crest Heights

For most floaters, the diameter of the column is so large compared to interesting wave lengths that a significant disturbance of the incoming wave field is to be expected – in particular for steep sea states. Two mechanisms are involved; diffraction (and reflection) and waves generated by the platform motions. Linear diffraction analysis with correction for wave asymmetry is believed to be accurate if we stay away from a zone of d/2 around the columns, where d is column diameter. Closer to the columns, non-linear diffraction should be considered. It will be difficult to avoid run-up induced impacts against cellar deck close to the columns and such events must be designed against.

In connection with model testing it is often difficult to obtain the amount of wave energy in the upper tail that is specified by the target spectrum. This problem is amplified when using the Torsethaugen spectrum due to the slower decay.

A Torsethaugen spectrum is used as target input spectrum for ALS model tests of a semi-submersible. From the calibration tests with no platform present, the target spectrum is compared with the average basin spectrum in platform position in Figure 16.

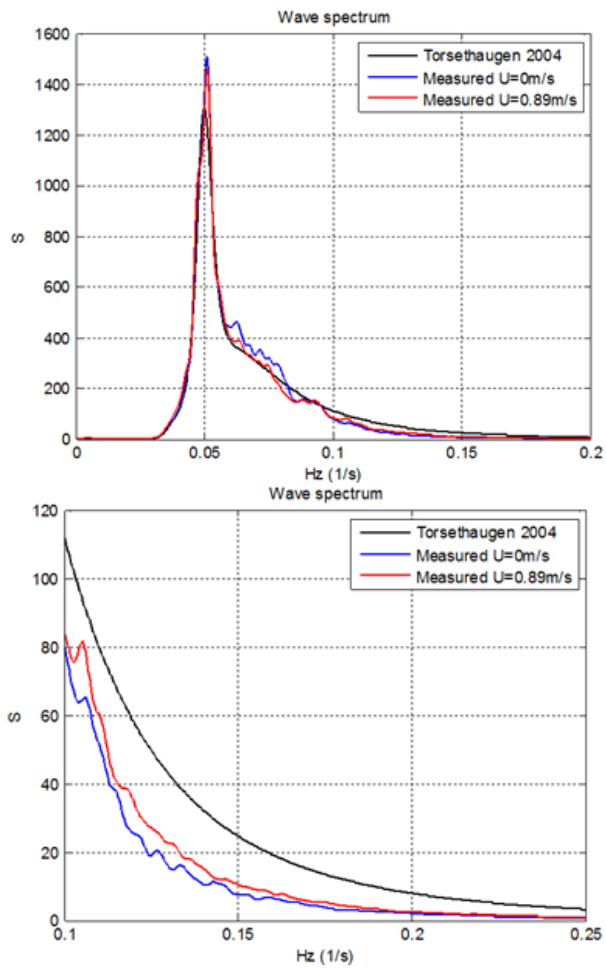

Figure 16 Wave spectrum for a sea state with H_s = 20.7m and T_p = 20.1s (ALS sea state), [18].

Tests are done with and without current. A reasonable agreement is seen except for frequencies above 0.1 Hz. Can this deviation effect the prediction of extremes for the relative crest height?

By numerical predictions using a linear diffraction code, MULDIF, the relative crest height (upwell) extremes around the platform are calculated both for the target Torsethaugen spectrum, black curve in Figure 16, and for the average spectra estimated from the calibration tests, red and blue curves in Figure 16, [18]. The linearly calculated crest height extremes are multiplied by an asymmetry factor 1.2 in order to approximate effects of non-linearity in the wave process. According to the results of Ref. [18], the worst relative crest height (upwell) extreme is typically 2-3m higher when using Torsethaugen than what is obtained using the average spectrum as input to the MULDIF analyses. The difference is due to the upper tail.

For large frequencies (short waves) the diffracted wave elevation transfer functions are large. For small frequencies (long waves) the transfer functions are small. When using a constant asymmetry factor for all frequencies, upwell will be larger for the spectra with a fat tail. However, the asymmetry factor is uncertain. Further work on the asymmetry factor is recommended. Further work is also recommended, regarding the high frequency ($f < 0.1$ Hz) shape of the wave spectrum for storm seas. What is the most accurate estimate, a theoretical wave spectrum decaying according to f^{-4} or the average model test approximation of the theoretical target spectrum? This is of some importance if the approach of a constant asymmetry factor is maintained.

Effects of Platform Motion

For an accurate assessment of airgap for a floater, the motions of the floater must be accounted for. The most important modes of motion are heave, roll and pitch. Slowly varying surge and sway may also be of some concern since they could have a similar effect as current. If current is of importance regarding airgap assessments, then these modes of motion may be important, [18].

The ability to move with the waves will typically reduce the required still water air gap compared to a jacket. If for example a still water airgap of 25-26m is required for a jacket to avoid wave deck impacts at 10^{-4} annual probability level (ALS consideration), a semi-submersible may avoid wave-deck impacts at the same probability level by a still water airgap around 20m, except close to the columns. This is an airgap much higher than the than the survival airgap, 13.5-18m, Ref. [20], of most drilling semi-submersibles. Of course, drilling semi-submersibles are not required to be checked for an annual exceedance probability of 10^{-4}, but the lower range will from the author's point of view also be exceeded at an annual exceedance probability of 10^{-2}.

In long period sea, airgap problems are typically non-existing since the platforms have time to respond to the wave. The challenge for floaters is steep seas where the platform has not enough time to raise the front. Steep seas are also the seas where non - linear mechanisms are most clearly pronounced. It is not only the effect on the crest height that is of concern, the crest front steepness may be equally important for floaters because this will affect the platform's ability to raise the parts of the platform facing the incoming waves.

The non-linearity in the incoming crest height is compensated for by the asymmetry factor, see e.g. [3, 18]. Non-linearity in crest front steepness is not that easy to account for in a simplified way. If one is to consider sea states including severe waves close to their limiting steepness, model test experiment is the best available tool.

A final comment related to the effects of motions is the consequences of combined seas, i.e. the local stormy wind sea is super-imposed on an incoming swell system. Such sea states can be a possibility for all types of wind sea since wind sea and swell sea are likely to be uncorrelated phenomena. But combined seas of concern for airgap assessments are primarily sea state along the steepest side on the ULS- and ALS contours, see Figure 15. The swell sea and wind sea will in most cases correspond to different direction of propagations. The wind sea is of course the dangerous sea regarding height of incoming crest, but if swell sea cause significant rolling independent of the incoming wind sea it may affect the available airgap in the corners of the platform. Short crested wind sea will have a similar effect on the rolling motion.

Effects of Wind

Wind should be included when assessing minimum airgap. It can amplify pitch and roll motions – especially if the corresponding natural periods are rather long. Some attention should also be given to cases where wind direction and wave direction are not the same. In extreme storms, a difference of more than 30° will be rare on the Norwegian Continental Shelf.

For floaters kept in position by dynamic positioning, the dynamic positioning should be included in the assessment of required airgap. If platform is heading into the wind sea and the wind direction is 30° off the wave direction, the resulting effect of wind force and compensating thrust force may add up to a larger pitching and/or rolling moment than one would have for the same platform and weather in a moored condition.

6 AIRGAP ASSESSMENT USING NORSOK STANDARD AND MARITIME REGULATION

The major difference between airgap assessments for semi-submersibles for production (following RR1) and semi-submersibles used for drilling (following RR2) is the difference in focus on ALS, i.e. 10^{-4} – annual probability metocean events. In Figure 17, the relative crest height (upwell) contours are shown for both ULS - and ALS sea state for a semi-submersible. The results are based on a linear diffraction code, WADAM, and a correction of the linearly predicted crest height by an asymmetry factor of 1.2. Long crested sea is applied. The difference varies over the projection of topside, but the ALS crest height is about 2 - 4m higher than the ULS results.

The observed difference between ULS and ALS should be well known to the rule makers. It is somewhat surprising that this large difference is accepted. Both production units and

drilling units are manned structures, both types of platforms are operating without any airgap related restrictions (except de-ballasting to survival draft for drilling rigs) on the Norwegian Continental Shelf. A thorough assessment on the consequence of the large difference on platform safety is recommended.

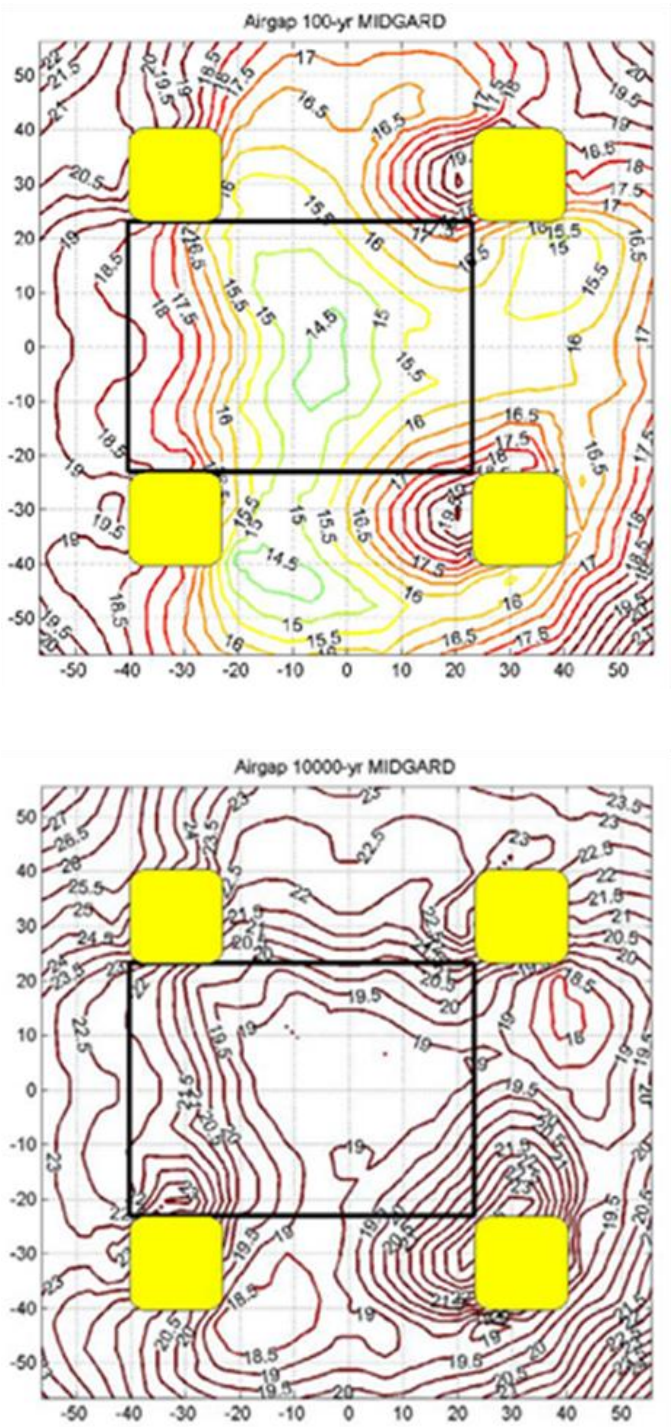

Fig.17 ULS (top figure) and ALS (bottom figure) relative crest height for a semi-submersible. From Ref. [18]

7 SUMMARY DISCUSSION OF UNCERTAINTIES RELATED TO AIRGAP ASSESSMENTS

Below is a discussion of what the author of this paper considers as the most important metocean sources of uncertainties regarding prediction of necessary airgap for fixed platforms and semi-submersibles. Recommendations regarding assumptions that have to be made during an airgap assessment will be given.

The recommendations reflect the views of the author and will occasionally go beyond what is recommended by "rule regime owners". The discussion is subjective and is primarily meant as a contribution to a discussion on how more robust still water airgaps can determined in the future – in particular for the semi-submersibles following the maritime regulation regime (RR2).

7.1 Fixed Platforms

Short Crested or Long Crested Sea

For fixed platforms long crested sea is likely to be slightly conservative. This author does therefore suggest that long crested waves should be used for estimating the ULS - and ALS still water airgap at the design phase. The conservatism introduced by that, can eventually be utilized in connection with future modifications of top side weight or worsening weather conditions.

Non Gaussian Sea Surface

For jackets and jack-ups, the airgap must account properly for the non-Gaussian structure of the surface process. The largest uncertainties related to the airgap problem is the adequacy of the second order wave theory. Can a second order crest height prediction be considered sufficiently accurate? Arguments can be given for both yes and no. Robust engineering therefore suggests that one should add a margin for uncertainties in the accuracy of second order theory. 5-10% margins on top of the ULS – and ALS predictions of crest height do not look overly conservative.

Area Effect

Another topic that causes some discussion is the area effect. This effect is a real effect. Nobody should be surprised by the fact that the largest crest within an area in the long run is larger than what is observed maximum in a single point for a dispersive random surface field. Uncertainties are, however, related to its magnitude. We will not discuss this further here, but 5-10% margins on top of the estimated ULS – and ALS – still water airgap do not look overly conservative.

If the resulting margin accounting for uncertainties in second order model and in the area effect is selected less than 15%, this should from authors point of view be verified by e.g. a properly designed model test.

Gravity Based Structures (GBS)

For still water airgap assessments of gravity based structures, what is said for jacket do apply, but in addition sea- structure interactions may an important contribution to the height of the still water airgap. A linear diffraction code may give

reasonable results except for very close to the platform legs, i.e. the run-up zone. But the surface field can be rather chaotic between the platform legs and one can question the validity of a linear assessment of the resulting maximum crest height. In particular, this should have focus for platforms with a rather shallow depth over the caisson. Linear analysis may be very conservative if wave breaking takes place as the wave enters the up-wave front of the caisson. The analysis may non-conservative if non-linear mechanisms are active over the caisson, but full breaking is not seen over the caisson. For such cases model test experiment can be useful.

A model test experiment was conducted in mid-eighties for the Statoil operated GBS Sleipner A. Based on test results, the incoming (undisturbed) 10^{-2} annual probability crest height was increased by about 5m due to the presence of the platform, i.e. an increase of about 30%. For the governing ALS event, the increase was merely 1.5m due to wave breaking, i.e. an increase of the incoming ALS crest height of 5-10%.

7.2 Semi-Submersibles

For semi-submersibles, it is more complicated to estimate proper still water airgap. For semi-submersibles operating at several drafts, focus should be on necessary draft in the survival condition. A floater is more robust against global failure due to wave-deck impacts than a jacket, but consequences can still be severe if a major impact wave-deck impact takes place, e.g. COSL Inventor at the Troll Field in December 2015. Another very severe case with a semi-submersible is the Ocean Ranger accident at the Hibernia field nearly 50years ago. A violent wave-deck impact caused rather severe local damages to the platform (e.g. broken windows and water into the control room). The impact itself did not directly cause the tragic outcome, but wave-deck was the initial event of what during next few hours escalated to a huge disaster.

Non Gaussian Sea

For air-gap considerations, accounting for the non-Gaussian structure of the surface process is as important for semi-submersibles as it is for fixed platforms. In practise this is often done by an asymmetry factor, see e.g. [3]. This may in most cases be sufficiently accurate regarding the correction of incoming undisturbed crest height.

The surface process under and near the platform will in severe wave conditions be rather chaotic due to diffraction and reflections. Linear analyses may give a good approximation in not too severe conditions, but will it be accurately enough in the most dangerous sea states regarding airgap assessments? This should be verified by properly designed model test experiments.

Sea – Structure Interaction

A challenge, in practise, can be to predict the effects of the platform motions on the estimation of required still water airgap. It is not the calculation of motion itself that represents the challenge. Linear theory can possibly be sufficiently accurate. The major difficulty is to select the governing combination of metocean characteristics regarding airgap assessments. Usually a rather simplified combination is utilized, e.g. pure wind sea characterized by H_s and T_p, which additionally is often assumed to be long crested. Various wave directions are checked. Turbulent wind is often used, but it is commonly taken to be in-line with wave direction. There may be a non-negligible risk that the simplified combinations will lead to non-conservative results due to under-predicted unfavourable motions of the platform. In order to improve on this and reduce uncertainties, the following cases should account for:

1. Short crested sea should be used for the wind generated sea. Main reason is to be sure that the effect of roll and pitch is properly reflected in the airgap assessment. One can use the spreading function shown in Eq. (11). The degree of spreading, n, could be taken as the value of n between 4 and 10 that is most unfavourable regarding airgap assessments.

2. If critical sea states can be of combined nature, i.e. wind sea superimposed on swell, one should simultaneously expose the platform to wind sea and swell sea. This can be done utilizing the Torsethaugen spectrum, but it is important to use a realistic directional difference between wind sea and swell (in view of the site of operation). The swell sea can be taken as long crested.

3. If rolling is important for airgap assessments, a case with the associated wind having a directional deviation of +/- 30° relative to the wind sea direction should be included to investigate sensitivity of roll to wind exitation. The sign should be selected based on what is conservative regarding rolling.

4. If platform is kept in position by a dynamic position (DP) system, the airgap assessment should include dynamic positioning and its effect on rolling unless is demonstrated that a DP system will not have a significant effect on rolling or pitching.

It is clear from above that a proper verification of sufficient still water airgap for a semi-submersible, in practice, cannot be done by numerical analysis. A properly designed model test experiment is called for. This does not have to be done for similar type of platforms, but this author recommends that at least one platform of each type of semi-submersibles should be exposed to such a model test experiment for the ULS level prior to receive class for winter drilling in harsh weather areas. This is beyond what at present is required for both rule regimes. For ALS level, swell is likely to be less important. Co-linear wind sea is also more likely to be acceptable. But one should assess the sensitivity of short crested modelling of the wind sea. Effect of DP system should also be assessed.

A model test experiment should include several sea states in order to identify the worst combination sea state. In addition, a rather large number (>20) of repeats using different seeds for the sea wave processes should be tested. This for the sake of capturing the short term variability of the airgap variable.

Finally, a good alternative would be to do a full long term analysis by exposing the rig to the 60-year hindcast data base,

NORA10. This data base gives estimates for simultaneously occurring swell and wind sea. This is conveniently done adopting a peak-over – threshold formulation of the long term analysis. The long term analysis must be done by numerical methods, but a limited test program might be necessary to verify transfer functions from a wave process to the airgap process and transfer function from wind process to airgap process. Maybe a linear assumption can be sufficiently accurate to indicate whether or not it is important to include swell and wind sea as separate system and to include wind induced rolling in the analysis.

8 CONCLUDING REMARKS

The paper is discussing the various contributions defining required airgap for fixed platforms and semi-submersibles. For jacket and jack-ups, the most important uncertainty is associated with incoming crest height. It is suggested that results of second order random theory are taken as a lower bound for point estimates. A margin should be introduced to account for a possible non-conservatisms associated with the second order random surface process. Area effect is important and should be included, and uncertainties related to this effect seem to be limited. For large volume fixed platforms, effects of diffraction need to be accounted for. If non-linear diffractions must be included, model tests are recommended.

Airgap assessments of semi-submersibles are much more complex. Required airgap can be estimated using numerical methods by accounting for non-linearity in the incoming waves by a wave asymmetry factor. Reasonable results can be obtained outside half a diameter from the platform columns. Airgap assessments should be validated by a properly designed model test experiment. Regarding a further discussion on what should be covered by a model test experiment, reference is made to the previous section of the paper

The paper demonstrates that an ALS requirement will require a larger still water airgap than an ULS assessment. In view of the difficulties in estimating accurate loads due to major wave-deck impacts it is recommended that a “no major wave deck impacts” at ALS -level should be adopted also for the Maritime rule regime.

ACKNOWLEDGMENTS

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3.3 Presentation & article by Jesper Tychsen & Martin Dixen

NTNU Norwegian University of Science and Technology

International Association of Oil & Gas Producers

Wave kinematics and hydrodynamic loads on intermediate water depth structures inferred from systematic model testing and field observations – Tyra Field Extreme Wave Study 2013-15

Jesper Tychsen, Structural Engineering Advisor
Maersk Oil, Danish Business Unit

The 3rd Offshore Structural Reliability Conference
OSRC2016
14-16 September, Stavanger, Norway

2016 Offshore Structural Reliability Conference

Wave kinematics and hydrodynamic loads on intermediate water depth structures inferred from systematic model testing and field observations…

Agenda:

- Introduction to study
- Field observations – Tyra – Denmark
- Framing of wave load design practices
- Irregular wave loading – what to expect
- Outline of extreme wave study 2013-15
- Key findings as relevant for structural reliability

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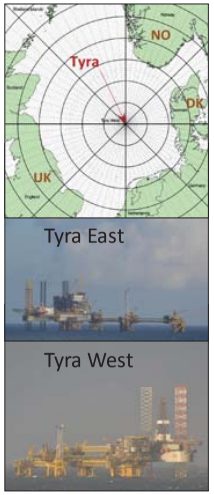
Wave kinematics and hydrodynamic loads on intermediate water depth structures inferred from systematic model testing and field observations…



Hs = 10m; Tp=14.5s and Sp=20

(approx. 20-30y return period storm conditions)

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Wave kinematics and hydrodynamic loads on intermediate water depth structures inferred from systematic model testing and field observations…

Tyra Design Basis 1999-2012:

- Tyra "East" (6 plf) and Tyra "West" (5 plf.) bridge connected
- North Sea offshore Denmark, d=45m
- 2016 seabed subsidence 5-6m – TWC 2016 air gap: 14m
- 1999 Phase I plan ("shutdown" & "strengthen" – jackets and decks)
- 2008 Phase II plan ("bridge reinforcement")
- Target: Safe production to 2025 (stricter ALS criteria than current code requirement applied)
- Design Basis:
  - 10,000y Stokes (umax= ≈10 m/sec. + "MO freak" 3D Umax ≈14 m/s)
  - 10,000y point max crest (incl. tide and surge): 16.5-16.7 m (ref. MSL)
  - 3.6m max. deck overwash – large WID testprogram – MO WID model
  - Focus on best possible resistance (pushover, FE, validation test, etc.)

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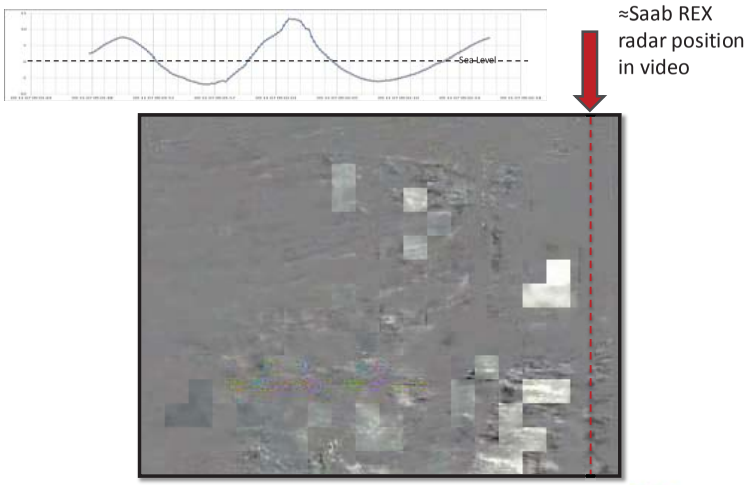
Wave kinematics and hydrodynamic loads on intermediate water depth structures inferred from systematic model testing and field observations…



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Wave kinematics and hydrodynamic loads on intermediate water depth structures inferred from systematic model testing and field observations…

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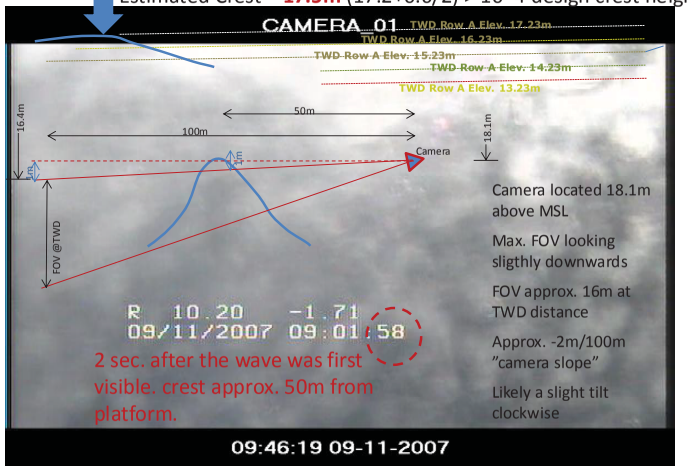
Wave kinematics and hydrodynamic loads on intermediate water depth structures inferred from systematic model testing and field observations…

Estimated Crest = **17.5m** (17.2+0.6/2) > 10^4 design crest height of **16.5m**



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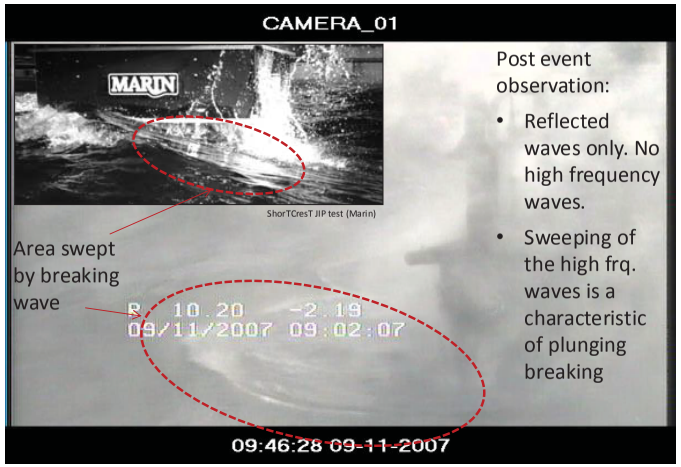
Wave kinematics and hydrodynamic loads on intermediate water depth structures inferred from systematic model testing and field observations…



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Wave kinematics and hydrodynamic loads on intermediate water depth structures inferred from systematic model testing and field observations…

Common Assumptions for Wave Load on Structures:

- Regular wave theory (e.g. Stokes 5th order) or NewWave
- Deterministic link from wave height to wave load
  - 100y return period wave => 100y return period wave load
- Normalised load statistics (e.g. hazard curves) for global mudline loads are generally representative for wave loads
- Collapse: Global load > Global resistance (1 DOF criteria)
- Hs: Only Long-term variable; Short-term: (Cr/Hs), (H/Hs)

Concern: If the occurrence of highly non-linear irregular extreme wave ("breaking") is not negligible, the above assumptions may be incorrect as a consequence of large transient and spatial variations in kinematics

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Wave kinematics and hydrodynamic loads on intermediate water depth structures inferred from systematic model testing and field observations…

- Initiated early 2013
- Target: "*Estimate the collapse probability of low air gap Tyra structures accounting for new knowledge*"
- *Multi-disciplinary project: Met-Ocean; Hydro-dynamics, Structural and Reliability*
- *Spring 2013 full team assembled:*
  - *Met-Ocean: DHI, Imperial College, Prof. G. Stewart*
  - *Hydro-dynamics: LIC Engineering, DHI; Prof. C. Swan (Imperial College), Prof. P. Madsen (DTU); Dr. H. Schäffer (Schäffer Waves)*
  - *Structures: Ramboll, Prof. G. Stewart*
  - *Reliability: Prof. M. Faber (DTU); Prof. J. D. Sørensen (AU); LIC Engineering*
  - *Wave flume and basin testing: DHI and Imperial College*
  - *Computational Fluid Dynamics (CFD): Validus Engineering*
- *Very high attention to consistency in simulation and assurance => quality of conclusions*

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Wave kinematics and hydrodynamic loads on intermediate water depth structures inferred from systematic model testing and field observations…

High level simulation approach:

- Expand 32y field storm record to "1 million y"
- Long-term: 1h sea states (H_s, T_p, S_p) + associated parm.
- Short-term: Semi-empirical fully non-linear irregular sea state model calibrated against dominating response
- 9 months of continuous wave basin and flume testing to provide sea surface and "response" statistics for calibration
- Monte Carlo simulation:
  - Realisation of all potential critical sea states in 1 million life-cycles
  - Identify largest extreme wave events
  - Stochastic realisation of all loads and resistance
  - Calculate if "load>resistance" in any failure mechanism
  - Result: Annual Pf in single mechanisms and for full jacket system

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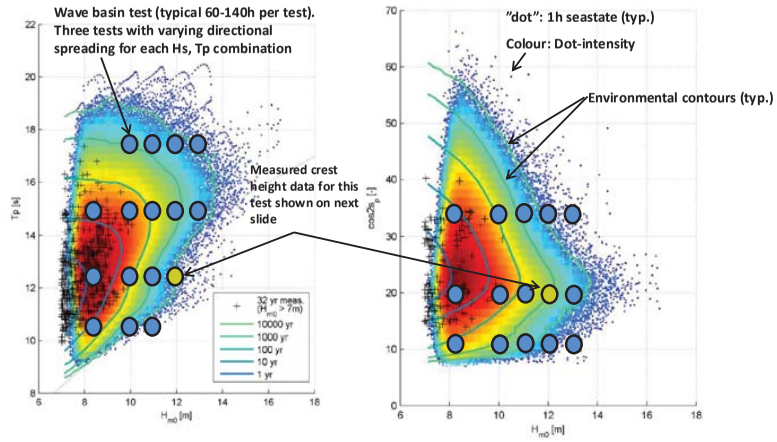
Wave kinematics and hydrodynamic loads on intermediate water depth structures inferred from systematic model testing and field observations…



9 months spent in wave lab! Long periods of 23h/day testing (1h for equipment calibration)

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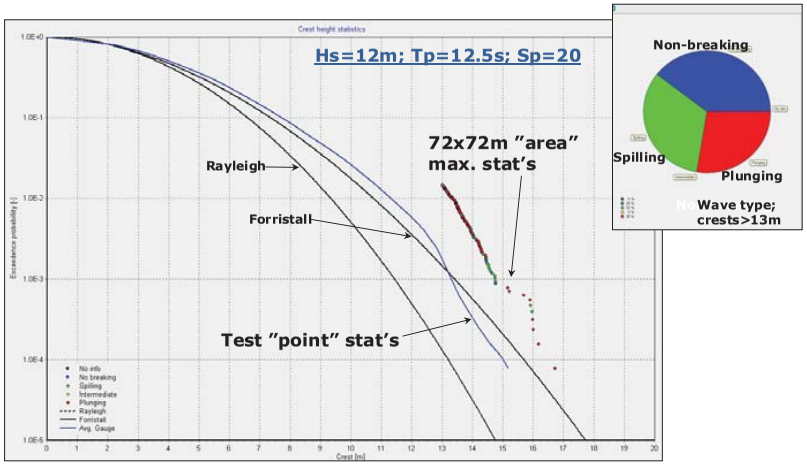
Wave kinematics and hydrodynamic loads on intermediate water depth structures inferred from systematic model testing and field observations…



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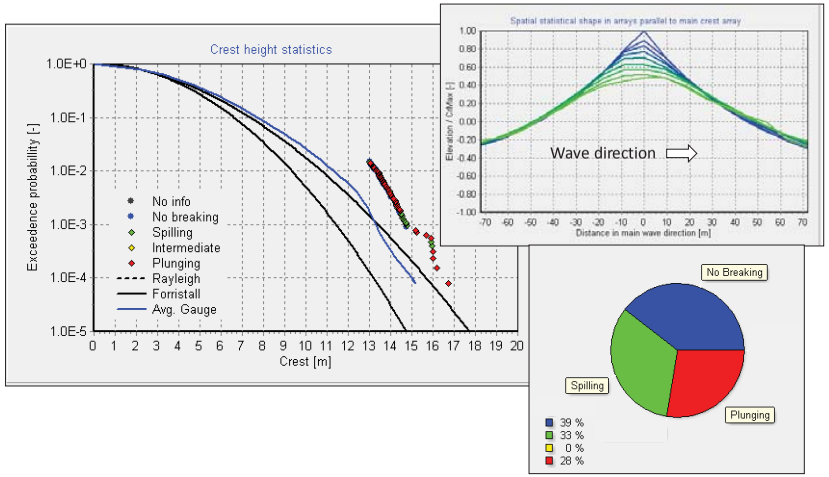
Wave kinematics and hydrodynamic loads on intermediate water depth structures inferred from systematic model testing and field observations…



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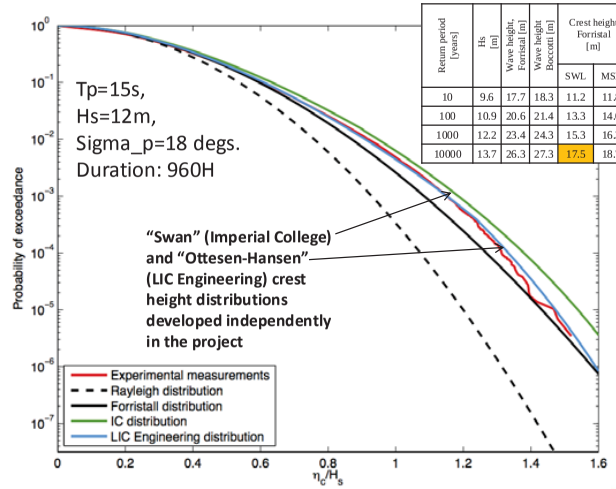
Wave kinematics and hydrodynamic loads on intermediate water depth structures inferred from systematic model testing and field observations…



Return period [years]	Hs [m]	Wave height, Forristal [m]	Wave height Boccotti [m]	Crest height Forristal [m]		Crest Height Swan [m]		Crest height Ottesen-Hansen [m]	
				SWL	MSL	SWL	MSL	SWL	MSL
10	9.6	17.7	18.3	11.2	11.8	12.1	12.7	11.8	12.4
100	10.9	20.6	21.4	13.3	14.0	14.4	15.2	14.1	14.9
1000	12.2	23.4	24.3	15.3	16.3	16.6	17.6	16.3	17.3
10000	13.7	26.3	27.3	17.5	18.7	18.8	20.0	18.5	19.7

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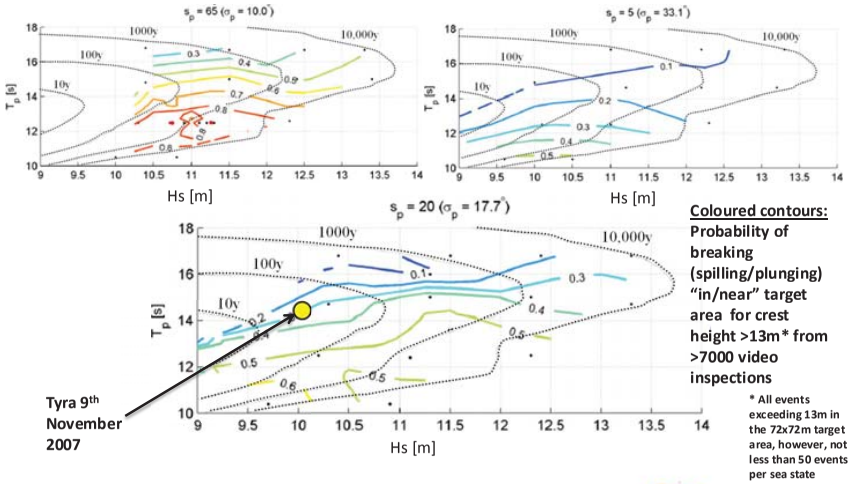
Wave kinematics and hydrodynamic loads on intermediate water depth structures inferred from systematic model testing and field observations…



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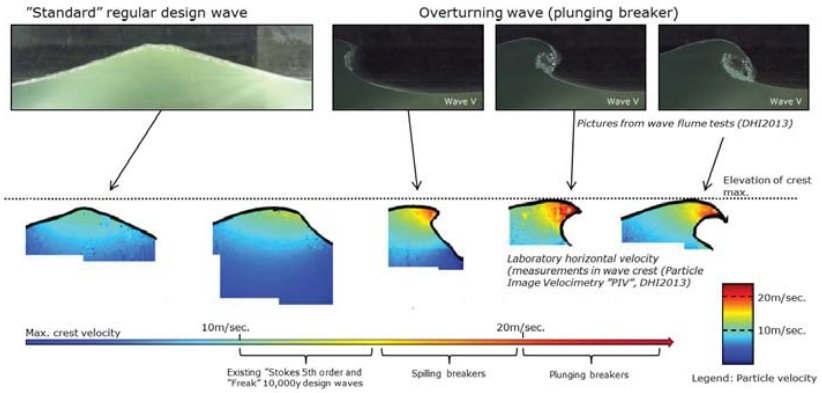

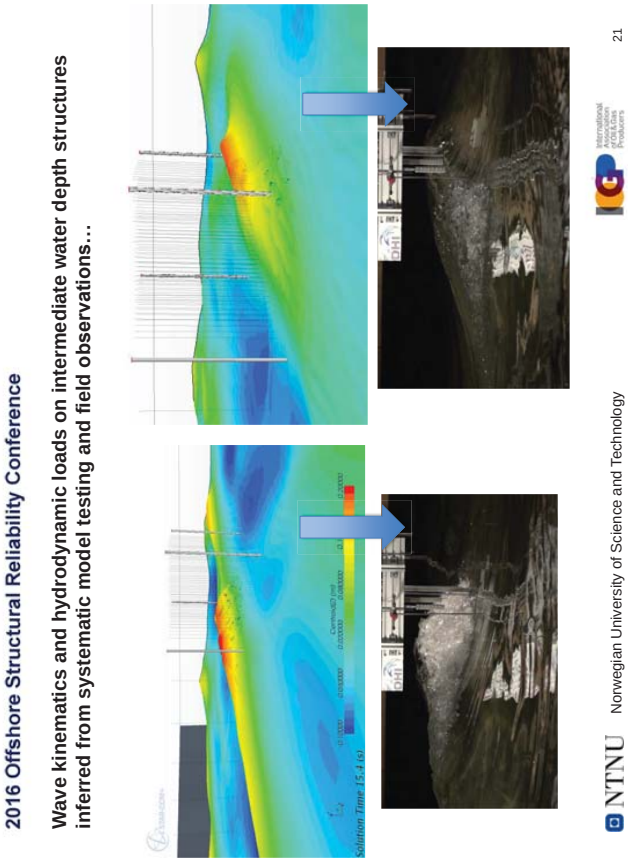
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Wave kinematics and hydrodynamic loads on intermediate water depth structures inferred from systematic model testing and field observations…
Solution Time 15.4 (s)
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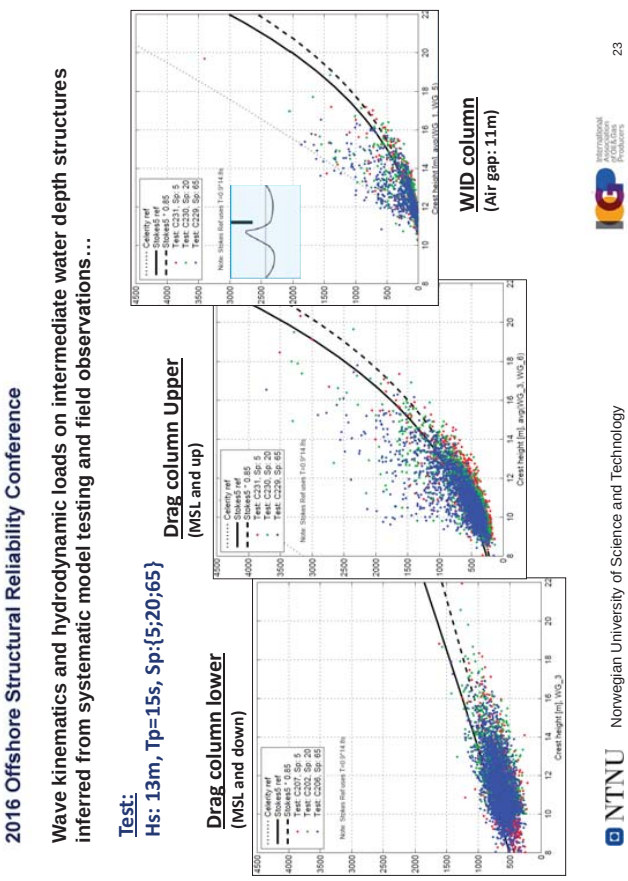
21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Test:
Hs: 13m, Tp=15s, Sp:{5;20;65}
Drag column lower
(MSL and down)
Drag column Upper
(MSL and up)
WID column
(Air gap: 11m)
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WID Column
Generic deck
Drag Column
Split at MSL
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Drag Column Load Comparison
2D events:
II: Regular
III: Mildly spilling
IV: Vigorously spilling
V: Plunging
Selected 3D events, long and short crested, non-breaking, spilling and plunging (equal selection)
3D
2D
Lower section [kN]
Upper section [kN]
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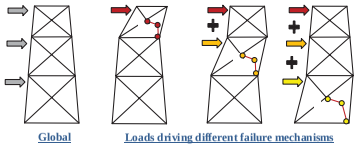
Wave kinematics and hydrodynamic loads on intermediate water depth structures inferred from systematic model testing and field observations…

Conclusions:

- 6% increase in (10^4y) crest height relative to Forristal
- Highly transient kinematics in extreme nonlinear waves
- Not a deterministic link between crest height and load
- Very high velocities in the crest of breaking waves
- 3D kinematics factor is elevation and event dependent
- Every 3rd to 2nd extreme wave (Cr>13m) is breaking
- The large spatial variations will challenge the assumption of "normalised global load statistics" being representative



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Wave kinematics and hydrodynamic loads on intermediate water depth structures inferred from systematic model testing and field observations – Tyra Field Extreme Wave Study 2013-15

Jesper Tychsen[1], Martin Dixen[2]
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ABSTRACT: In 2012, a photo was discovered of a plunging, breaking, extreme wave, occurring in a 10m significant wave-height storm at the Danish Tyra field (depth≈45m). A search for further data/evidence revealed a video showing a second, even larger, plunging breaker (crest height estimated at 17-17.5m) in the same storm at the same field site. These observations were highly concerning as they exceeded the existing 10,000 year return period abnormal event design criteria with respect to both crest height and crest kinematics. The estimated return period of the storm was only 20-30 years. This triggered an extensive study in the period 2013-15 aimed at mapping highly non-linear extreme wave events and clarifying the impact of these in relation to the integrity of structures in the Tyra field. This paper is primarily concerned with setting the scene for the re-assessment studies and reporting the extensive works to map the hydrodynamics and wave load ("response") statistics of extreme non-linear irregular waves. Due to the lack of a fully non-linear wave theory, the work builds on wave flume and basin testing as the tool to investigate the short-term statistics of the Tyra extreme sea-states. The results point to important differences between the assumptions underlying standard industry practices and the statistical nature of highly non-linear irregular extreme wave events, both in terms of the crest height and its 3D shape, as well as assumptions for particle kinematics and related loads. An OSRC 2016 companion paper focuses on how the developed knowledge has been incorporated into a Monte Carlo simulation model and presents the overall results.

KEY WORDS: Irregular waves; Breaking; Wave kinematics; Global wave loads; Hydrodynamics; SRA

1 INTRODUCTION

The Tyra gas field is a hub for gas export and approximately 90% of Danish gas production passes through it. The first structures were installed in the field in the early 1980s. Today, 11 bridge connected platforms are operating at two field locations (Tyra "East" and "West", 3km apart). Owing to depletion of the reservoir rock, the Tyra field is prone to seabed subsidence. Presently, the seabed has settled approximately 5m at both field locations. In year 2000 the first platform exceeded the predicted 10,000-year return period design air gap. This triggered the Tyra Subsidence Phase I re-assessment project. This project was followed by a second phase targeting the integrity of riser systems and interconnecting bridges.

Key points in this initial (re-)assessment project were:

- Design check for Ultimate and Accidental Limit States (ALS).
- The ALS employed a regular Stokes 5th order design wave based on the predicted independent 10,000-year return period crest height (16.5-16.7m including tide and surge with crest peak particle velocity of approx. 10m/s). In addition a check was included for a "worst case" 3D "Freak Wave": a deterministic event with 14-15m/s peak crest velocity was defined for this. Design waves were derived from site measurement records dating back to the early 1980s.
- Structures were strengthened to meet these criteria: addition of diagonal bracing to portal frame module support frames; some deck reinforcements; and grouting of compression bracing. This enabled in the order of 10MN horizontal wave in deck (WID) loading to be resisted.
- A large test program was performed at scale 1:26 to develop WID models applicable to the (typically open deck) Tyra structures (grating covered H beams).
- Platform deck structures were strengthened to meet at least a 3.6m deck inundation in the design WID event.
- The strengthening was supplemented by adverse weather production shut down and depressurization of equipment and prohibited access of personnel to lower decks.

Until 2012 these re-assessment criteria were considered prudent and conservative relative to general industry practice and code requirements. However, further to the discovery in 2012 of a plunging breaker in the Danish Tyra field (Figure 1, d≈45m), followed by a video of an even larger plunging extreme wave in the same storm that exceeded the predicted 10,000-year return period design crest height, we were concerned that our assessment criteria were not adequate in terms of both the statistics of the extreme crest heights as well as extreme wave particle kinematics. This led to the initiation of two new projects: Tyra "As-Is", targeting an as accurate as possible estimate of structural reliability considering "New Knowledge" and Tyra "Future", targeting a permanent solution in the event that the original design basis was found to be inadequate.

The present paper provides an overview of the work related to "New Knowledge" with respect to extreme wave loading performed over the period 2013-15. A companion paper [1] presents the overall structural reliability analysis (SRA) work and general findings of the Tyra "As-Is" study.

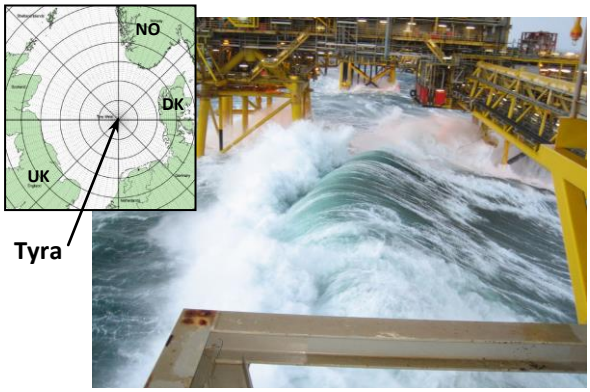

Figure 1. Tyra field, Danish North Sea. H_s=10m; T_p=14.5s; d≈45m. Insert: Field location (grid 100km).

2 FRAMING THE SCOPE OF THE INVESTIGATION

The core topics of the present paper are to present details of work related to kinematics, loads etc. related to highly non-linear sea-states. However, beforehand, we need to spend some time framing the boundaries for this investigation. This is an especially important issue, as we now believe that in the past we have not given this sufficient attention and relied too much on industry practice and code guidance.

2.1 Reliability of Offshore Structures – a Multi-discipline Challenge

Assuming our ultimate need is a design methodology for load and resistance which covers a variety of different offshore structures and wave-load driven failure mechanisms, we need a high level of technical knowledge within a number of disciplines to succeed:

- Met-ocean: Determine the long- and short-term environmental statistics, i.e. sea-state (significant wave height, H_s, peak period, T_p, sea-state spreading, s_p) as well as individual wave and crest height statistics.
- Hydrodynamics: Deliver the models for water particle kinematics throughout the water column (x,y,z) in extreme waves as a function of time and the models converting these kinematics to structural loads.
- Structures: Determine the structural response to given static or dynamic loading events. Assess failure mechanisms and their resistance/failure load.
- Reliability: Combine the above knowledge to ultimately assess the annual probability of a given major accident hazard. Within the SRA it is very important that consistency in "consequences" relative to "failure" probabilities is defined. Furthermore, the reliability experts must duly account for the relevance and accuracy of the other discipline models applied.

It is generally accepted by the different discipline communities that large discipline challenges exist, but less common is an in-depth discussion of the technical challenges present across disciplines. An important example of the cross-discipline challenge is between "hydrodynamics" and "structural" disciplines.

To illustrate this we will look at a standard space frame jacket. This structure, and any other structure, will in broad terms, collapse at the time the "load" exceeds the "resistance". In the past, SRA models have been built around a format that collapse occurs when "global load" exceeds "global resistance", i.e. a single failure mechanism format, equal to that of a simple tension rod. This assumption of a single failure mechanism is convenient, as it allows application of relatively simple SRA methods. The question is whether we can simplify a space frame jacket into an equivalent tension rod without any sacrifice?

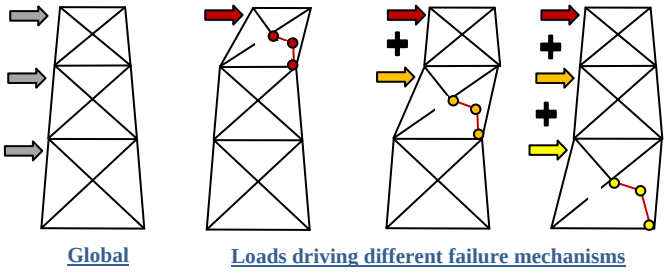

Figure 2. Space frame jacket failure mechanisms. Subsets of global loads driving different failure mechanisms. Only typical bracing failure modes shown. Same principle applies to leg failure modes.

A space frame structure is built of trusses. Each truss is strong in tension and compression and weak in bending. Building in triangles, all loads in the trusses can be transferred by compression and tension giving a strong, light-weight, structure. Applying a load far lower than its capacity, a steel structure will respond linearly, similar to an elastic spring. It is straightforward to calculate the loads in the trusses. Knowing the capacity in tension, compression and bending of trusses of given dimensions and the capacity of nodal joints, we can determine the load level where the first truss fails. This is the "first member failure" capacity. If a structure is statically determinate, it cannot be loaded beyond the first member failure load. If the structure is redundant, alternative load paths exist and depending upon the deformation capacity of the failing member(s), these alternative paths provide additional system capacity beyond first member failure. In this case the structure collapses once a failure mechanism is developed as sketched in Figure 2. As a jacket is cantilevered from the seabed, force flow in the diagonal braces will be from the top and down. It follows that a given bracing failure mechanism will only be driven by the loads generated on the structure above the location of the failure mechanism; so, except for foundation failure (or leg failure if the piles go un-grouted through the legs), only a subset of global loads will drive jacket failure mechanisms.

Jacket structures are thus much more complex than a single tension bar. Many failure mechanisms located at different locations can lead to global collapse of the structure. If the designers have truly optimized the structure, each of these failure mechanisms will occur simultaneously when the design load is reached (assuming the loading on the structure and the resistance of the mechanisms is exactly known).

If we now consider the case where the resistance is a stochastic variable and the structure has been optimized such

that there are 20 potential failure mechanisms, the system capacity will be determined by the mechanism which has the lowest realization of the resistance, i.e. the lowest of 20 realizations. The situation is similar to a multi-link chain. Of course in a real structure all failure mechanisms will not have the same deterministic capacity and the effect will be less dominant. Nevertheless it is non-conservative to assume a "single link in the chain only" (single failure mechanism model). But more importantly, to be able to determine the system collapse probability, we need to develop a method which can trace the load statistics in each of these failure mechanisms (and the correlations between mechanisms) to be able to obtain the system collapse probability correctly. Thus global loads will rarely drive global failures. It follows that if we want accurate estimates of collapse probability, the met-ocean and hydrodynamics disciplines need to develop models which can accurately predict the statistics for particle kinematics (and loads derived from these) for many different combinations of "wetted areas driving failure" throughout the water column.

Once realizing the above it becomes ever more apparent that regular 2D wave theory (or equivalent linear focused NewWave type of deterministic events) will be challenged as the local variations in non-linear irregular wave kinematics especially high in the water column cannot be matched correctly by such waves.

Of course, if the irregular wave kinematics are always lower than the kinematics in the equivalent regular wave and linear NewWave throughout the water column, it would be conservative to apply regular design waves. But is it so? When plunging occurs, we know the particle velocity in the crest exceeds the celerity which, from measurements, is a relatively well-known and stable parameter. Taking the two Tyra breaking wave observations, the celerity of these is expected to be around 16-20m/s. In plunging breakers, peak particle crest velocities may exceed the celerity by up to 20-30%. For Tyra this means particle velocities in the order of 20-25m/s might have been present in the observed plunging breakers. Recalling the design basis extreme crest kinematics of 10-14 m/s, we have strong indications of potential failure modes high in the water column that may be significantly affected by highly non-linear events and, unless these events are extremely rare, they will play a role in the reliability of offshore structures.

It can rightfully be argued that shifting from regular to fully non-linear irregular wave models will lead to a disproportionate increase in analysis complexity: not only requiring us to apply more complex new irregular short term models, but also to update the long term statistical models since parameters in addition to H_s will be required to define the storms. However, as we were concerned that irregular load effects may be the most important parameter for the reliability of many structures/-failure mechanisms, our conclusion was that we indeed needed to take on this more onerous task and try to clarify the effect.

This leads to a key issue: how does the offshore industry control the reliability of structures today and what are the potential shortcomings?

For the commonly applied codes for offshore structures, it does not seem to be a recognized general practice to support the code design procedures with a definitive SRA model. Consequently, there are many instances where there is lack of clarity to the code user regarding the safety level a code compliant structure will have. These codes are primarily "performance" based, meaning either calibrated to match the safety level in older code revisions or to match the safety level of structures which have shown a satisfactory performance.

An exception is the work in the UK approximately 15 years ago [8, 9] to develop a North Sea Annex to the otherwise performance based ISO 19902 code for fixed offshore structures [7]. The SRA model principle applied [8, 9] is largely based on work led by Shell in the 1990s. The overall reference for this is given by Efthymiou et al. [2], re-published in 2011 [3]. Important model background information can be found in references 3 & 4. As reference 9 does not give a complete description of the SRA model, in the present work we focus on the original Shell work, which is referenced in appendix A.9.9.3 of ISO19902 [7].

As it is relevant for setting the scene for the irregular wave load topic, a short summary follows of a few important issues relating to previous SRA studies and code development:

- A common assumption inherently included in design codes is a deterministic link between wave height and wave load (on a given structure). The "link" is established by assuming 2D regular wave theory (typically Stokes 5th order or equivalent linear focused NewWave theory, or similar), can describe the wave kinematics and for these kinematics, design loads in the structure are calculated applying a code prescribed standard load model.
- With this deterministic link between wave height and wave load, the wave load statistics become a function of the wave height statistics. I.e. the 100-year return period wave generates the 100-year return period wave load and similarly for the 10,000-year return period wave.
- Having the 100 and 10,000-year return period wave loads, code developers have what they need to set standard code design criteria. The 10,000-year return period wave is used in the accidental limit state ("Abnormal Environmental Event" AEE) to ensure a minimum annual safety level around 10^{-4} (if a characteristic resistance is used). In the Ultimate Limit State (ULS) check a safety factor is applied to the 100-year return period load to ensure the design is safe. The safety factor format is dependent upon the structural analysis format (linear "first member failure" or fully nonlinear "pushover" type). For example, ISO 19902 specifies that for pushover analyses of a normally manned platform, the ultimate system capacity of the structure (total collapse load) must exceed 1.85 times the 100-year load (RSR=1.85).
- Even though the SRA work in references 2-5 is not the official ISO SRA model, it matches the requirements specified in the code: the RSR value of a "High Safety Class" structure of 1.85 can be found in references 2 & 3. This fact makes the SRA model in this reference important; as well as that in reference 8 where, using a similar model, it was concluded that the partial safety

factor on wave loading of 1.35 in ISO 19902 [7] was conservative for the UK North Sea.

- In terms of wave loading statistics it is important to note that the model outlined in references 2-5 is based on:
  a. The assumption that normalized mudline load statistics (global loads) on a vertical "stick" (tube OD of 1m) are a representative measure of loads driving all global failure mechanisms. This is also an assumption in references 8 and 9.
  b. The assumption that regular wave theory (or similar NewWave, etc.) links wave height with global mudline load effects.
  c. A wave load model that is built around the long term statistics of the most probable short term (storm) wave height. The short term load variation is based on the theoretical coefficient of variation (COV) of a drag dominated load process in deep water linear sea-states: i.e. no higher order irregular kinematic effects seem to be included. It is not clear if any wetted area effect above MSL is included.
  d. The long term distribution of global loading is assumed to follow an exponential distribution (this gives linear hazard curves in log-scaled return periods).
  e. A first order SRA model which is applied to calculate the annual collapse probability. A single failure mechanism is considered ("global load" exceeding "global resistance"). For the purpose of obtaining an analytical expression for the reliability, the 20-year maximum global load is assumed to follow a lognormal distribution (duly fitted to the exponential distribution) and the resistance of a fully developed failure mechanism is assumed lognormal distributed too with a COV of 0.05.

It is clear that to implement irregular waves consistently in an SRA model we cannot apply the existing industry model outlined above. Assumptions of global load driving global failure, that regular or similar wave theory is appropriate, that short term linear deep water sea-state load variations and statistical models for single failure mechanism systems are applicable, are all conflicting with the variable nature of irregular loads.

2.2 In-situ Field Wave Records

To complete the framing we need to discuss what we actually know about the extreme waves acting on our structures. We put a relatively large effort into in-situ environmental monitoring, but what do we actually measure and maybe more importantly, what do we not measure? I.e. where do we use theory to provide knowledge instead of direct measurements?

The wave monitoring systems we deploy offshore (radars, lasers and buoys) have generally one thing in common: they deliver an elevation time series of a "point" at the sea surface. From this time signal we can directly determine the wave period and the wave and crest height for single waves. Looking at a longer time-span we can derive the significant wave height, peak period, tide, surge, etc. Conveniently this is exactly the data we need to feed into the standard industry design procedure: to generate a regular wave we just need knowledge about the wave height and the wave period, and, as the long-term statistics are solely based on knowledge about H_s, we also have what we need to support long term statistical models. Hence we are satisfied with our approach as we get the data we need to feed into our standard design procedure. But, on the contrary, do we get any warnings if anything is wrong with our assumptions? In other words, what is really interesting is what do we not measure:

- A time elevation signal does not tell anything about the shape/geometry of the wave being measured. We know little about the location of the measurement relative to the crest 3D peak, the decay along the crest, or anything about the steepness of the wave. We do know the relative steepness difference between the front and back of the crest, but not the actual steepness.
- It follows that in 3D sea-states, point surface elevation records will miss the true 3D crest peak. Looking at an area around the measurement point, crests will generally be higher. The larger the reference area the larger the chance of finding the true and larger 3D crest peak. Due to lack of measurements the 3D area effect is assessed theoretically or by basin testing.
- A point surface elevation record will not tell the type of wave, i.e. no information in the signal whether the wave is spilling or plunging breaking.
- We may miss the true crest peak due to having a too crude logging frequency. Many historic wave records are based on a 2Hz logging frequency. The steeper and more narrow the crests (highly non-linear irregular events) the higher the statistical sampling error due to too few measurements. The effect from a 2Hz logging frequency in a steep non-linear sea-state is assessed later.

Once we recognize that detailed knowledge about 3D non-linear irregular sea-states is important for the integrity of our offshore structures, we must also be aware that our standard offshore wave monitoring systems do not deliver much information (if any) which can assist in the revision of assessment methodologies. [Note: to mitigate the identified shortcomings in the longer term and improve our future offshore extreme wave monitoring, a proto-type development of a new wave scanner has been initiated. Based on data fusion of lidar point clouds (570,000 points measurements per second), long- and short-range infrared cameras and a standard optical camera, the target of the new scanner is to provide accurate large area 3D surface time series and surface foam intensities (for breaking detection).]

For the present work, we need to develop new knowledge from sources other than offshore records. We should not only look carefully at wave monitoring, but also take a look at the monitoring results used to support our load models. In the following we will take a look at some of the data utilized to support the choice of our load models. However, since this topic is not a main focus of the present paper, only principal issues are addressed.

Load monitoring programs performed for two North Sea jackets: Tern [6] (≈20,000t jacket, d=146m, monitoring during a 3-4 year period in the mid 1990s) and Magnus (≈40,000t jacket, d=186m, monitoring approx. 3 year period in the mid

1980s) play an important role in the development and validation of the industry standard load models. Particularly the Tern program has been a major source of data. Both jackets were instrumented in the mudline area close to a leg corner. Hence both monitoring programs were targeting global loads. These two jackets are amongst the largest platforms in the North Sea, installed in the deepest waters.

As the validation/development of the load model largely has been performed against global loads on very large space frame structures, the validation will be performed against "the sum of loads" on many members, and, as the structures are large and in deep water, a significant amount of this global load is generated deep in the water column. The sheer size of the structures and the large number of members are inherently a filter on the load process. Thus any locally more onerous load model representing only a subset of the loads can be overlooked simply as it may vanish in the total load sum. This is a concern as we apply the same load model for much smaller structures in shallower water depths and in many cases where failure mechanisms will be driven by a subset of the loads only. Consequently, are we sure the validation against data records from structures such as Tern and Magnus justifies general application? This concern is somewhat magnified by a comment in reference 6, which states that the Tern data is low pass filtered at 0.3Hz. Even though the jacket is large, low pass filtering at such a low frequency leaves little chance that any higher order deviations survive the filtering: it can be feared that what seems to be an aggressive filtering setting has removed parts of the wave loading.

3 THE TYRA "AS-IS" SAFETY WORKSCOPE

Once we reached the conclusion that our original design basis for Tyra assessment was challenged by new knowledge, the Tyra "As-Is" Safety project was launched early 2013. The scope was to determine the collapse probability of Tyra Structures considering "New Knowledge" on extreme wave loading.

Realizing that we faced many and complex non-linearities and to avoid any conservatism, a Monte Carlo based simulation approach was chosen. The principal work flow in the simulation is:

1. To achieve convergent results to a 10^{-4} to $3x10^{-5}$ annual collapse probability level, prepare a long-term simulation model which can generate the 1hour sea-states (H_s, T_p; s_p) occurring in a "1 million year" period.
2. Develop models which can generate all associated stochastic data during each 1hour period: tidal current (magnitude and direction); wind driven current (magnitude and direction); surge and tide; wind speed, turbulence and direction, etc. That is all parameters which influence the load vector. Model uncertainties, subsidence forecast etc. are also included properly in this approach. In principle all parameters can/are treated as stochastic variables. This approach ensures any conservatism in load contributions from associated load parameters is avoided as far as possible; it just requires we can describe the distributions and correlations.
3. Develop a wave simulation model conditional on sea-state which is sufficiently accurate to generate the maximum response on each structure within each hour. Due to the low air gap situation the model development should target accurate predictions of WID load statistics.
4. For each structure develop a new response model which accurately allows evaluation of load statistics for each of the possible failure mechanisms in each structure. The capacity of each failure mechanism in the response model is calibrated using a series of detailed pushover analyses of each structure. Further details are included in reference 1.
5. Due to the low air gap situation, failure was conditioned to a WID event larger than 0.5m deck inundation. We could only allow this assumption as the Tyra structures previously have been heavily strengthened to meet WID loads typically in the order of 1000T, and it would take a significant WID event (>0.5m) to collapse the structure. A first pass screening was performed to identify the sea-states which had the potential to generate a WID event. The screening was selected as: H_s > (air gap + 0.5m – tide –surge)/1.7. I.e. it was assumed that events with crest height/H_s > 1.7 would be rare and we could thus screen at sea-state level. Several hundred thousand 1 hour sea-states passed this first screening criterion. From here the following steps were performed using Cloud based computing.
6. The surface elevation throughout each of these 1 hour sea-states was generated by a fully non-linear irregular semi-empirical sea-state model (as developed in the project) and in a second screening it was checked whether any wave in the generated sea-state reached the actual deck structure within the 1hour duration. If so the actual seed was stored and the sea-state marked. At this time we have a file of all WID events with inundations larger than 0.5m in any of the deck structures/bridges in the field.
7. For each of these events the final step was to make full realisations of the kinematics field at the time of the event and all associated parameters, model uncertainties, subsidence forecast, structural resistance, dynamic response, etc. etc. and to step all of these through the detailed response model for each structure. For each time step we check whether the loading on any failure mechanism in the structure exceeds the resistance.
8. The end result is now a file of all 1 hour sea-states generating a WID event occurring in a 1 million year period and for each event we know if any failure mechanism in the structure has collapsed.
9. The annual failure probability is now a simple count of the number of years containing one or more collapsed failure mechanisms divided by 1 million.

Further details are included in reference 1. The above approach is virtually an explosion in complexity relative to existing SRA models [2] and code based procedures but the approach ensures consistency across all the "boundaries" and "disciplines" as discussed in the framing. To limit the number of uncertainties a crude Monte Carlo approach was deliberately applied. That is we worked our way through all potential critical sea states simply to avoid introducing further uncertainties due to a more "intelligent" simulation set-up. However, this did not come for free. At the time this was the largest cloud computing job running in Europe.

In the remaining part of the present paper we will look into the work undertaken to establish the basis for the sea-state model development. We will focus on general findings, i.e. findings related to extreme events in highly non-linear irregular sea-states which may be applicable far outside the Tyra area water depth.

4 BASIN AND FLUME TEST PROGRAMS

The main reasons why breaking of extreme waves is not part of industry procedures are probably because standard wave monitoring equipment cannot measure them and no suitable non-linear wave theory can predict them. It is for this latter reason that it is not possible to make a theoretical prediction of the statistical occurrence of breaking events or to determine the statistical impact of these in relation to loading. Following the recommendations/conclusions from the recent CresT and ShorTCresT JIPs [10, 11], wave basin testing seemed to be the only option available to obtain the required data on highly non-linear and irregular extreme wave events. Thus the methodology selected was to develop a semi-empirical approach: perform tests throughout all sea-states which could generate a wave reaching the deck and then calibrate a semi-empirical simulation model to match the test results and as part of this duly account for model uncertainties.

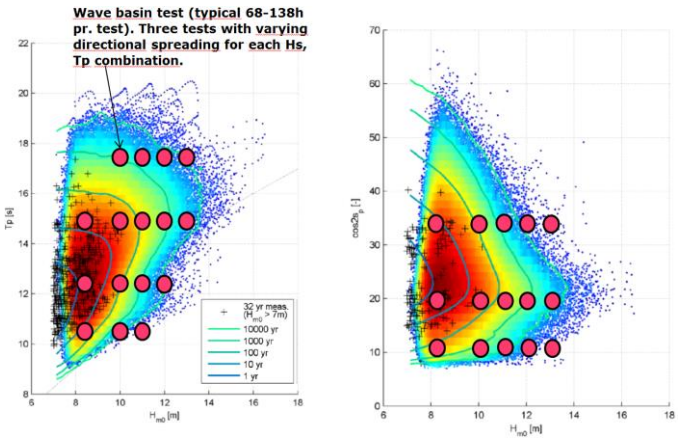

Figure 3. Series "A" sea-state core test program (red dots). Small dots are 1h sea-states in a 1million year period. Colour indicates "dot density". Crosses are 1h sea-states in the 32y long offshore record available. Lines are env. contours. Left: (Hm0,Tp). Right: (Hm0,cos2sp).

The main test program covered continuous testing through an 8-9 months period from summer 2013 to spring 2014 in the DHI shallow water basin and flume in Hørsholm, Denmark. In parallel an independent assurance program was performed in basin and flume at Imperial College (IC) in London. All tests were performed for a full-scale water depth of 45m. The test scale was generally 1:90 at DHI and 1:72 at IC. The initial 1-2 months of testing at DHI were spent validating and mapping the basin performance, optimal wave generation, wave gauge performance, sampling, etc. After this, a number of test series were performed. The two most important series were the "A" and "C" series. The "A" series targeted a large program testing a grid of sea-states spanning the upper tail of the Tyra environmental contour. The "A" series target was to establish the undisturbed sea surface statistics, i.e. crest height, 3D shape, breaking probability, etc. in the absence of a structure. 52 sea-states with varying combinations of H_s, T_p and s_p were tested, see Figure 3.

The target area in the basin measured 72x72m full-scale. Initially the area was covered by a grid of 5x5 standard "rod type" wave gauges. However, as there was concern about the performance of these gauges (run-up and air draw down) in steep and overturning waves, it was decided to replace these with a grid of 9x9 wire gauges inspired by the gauges applied by Imperial College in the CresT and ShorTCresT JIPs. A rerun of the most important sea-states was made to ensure full coverage of the relevant sea-states with the 9x9 grid. From Series "A" we have elevation records in a 72mx72m area with 9m spacing. Breaking detection was performed visually based on video records. All events in Series "A" exceeding 13m crest height in the target area, but not less than the highest 50 events (more than 7000 in total) were categorized by video inspection. An application, ePlot, was made to visualize all test statistics including crest height exceedance plots, celerity, normalized shape development, breaking probability, area distributions and crest volume above threshold. For each categorized event the ePlot application also includes a direct video link.

In short, the Series "A" test results support the development of the semi-empirical wave model for all statistics which are related to the sea surface. Series "C" tests are basically a rerun of all the Series "A" tests but instead of the grid of wave gauges, simple "instrumented response models" are located in the target area. The concept of testing "instrumented response models" instead of scale models of "true" structures requires some explanation.

Scaling (Froude) of the important parameters related to the sea-state (surface, kinematics, etc.) is known to work well for common lab conditions. That is the link from the lab to full-scale is reasonably reliable for the sea properties (and the best we have as no closed-form theory applies). Scaling of lab recorded loads is more problematic. Load processes which are Reynolds number dependent, for example, are problematic, as well as processes which somehow involve significant sea to air effects. It is commonly accepted that sharp corner profiles, such as deck structures, can be tested in lab-scale and the recorded loads can be reliably scaled to full scale. This has been utilized in many industry WID assessments in the past, where the approach is built around testing a detailed model of a topside structure in a few selected high return period sea-states. From this, using a few statistical assumptions on sea-state reoccurrence period and the fractile of design response, the 10,000y return period WID load is derived.

With reference to the discussion in the framing of our problem, a main concern is that the short term variation of 1 hour maximum loads in fully non-linear irregular sea-states for many structures/failure mechanisms may be significantly larger than generally assumed. A consequence of an increased short-term load variation is that less severe sea-states may contribute to the extreme load distribution. That is sea-states which are normally not considered critical may become

critical if the variation of short-term loads increases. It follows that methodologies based on testing a few 10,000-year return period sea-states may be challenged, as the 10,000-year return period loading may be generated by rare events in much more common sea-states. This is the reason for selecting a large test program, Figure 3, with testing of sea-states down to approximately 10-year return period.

With 11 jackets and 7 bridges in the Tyra field not meeting air gap requirements, it is physically impossible to test scaled geometric models of these throughout all required sea-states. Even if we had chosen this approach, the use of complex structural and load models would blur conclusions as we would be left unsure whether the statistical effects were a consequence of the load model or the kinematics.

Spending a little time rethinking the challenge and asking ourselves what is our real concern, we concluded that although challenges may exist in general for load models, for Tyra we were less concerned about this issue, as we had 15 years of WID tests at scale 1:26 and a detailed semi-empirical WID load model based on significant deck component testing. Hence, our main concern related to the statistical variation of fully non-linear irregular wave kinematics. Hence, planning the work we should focus on a best possible "isolation" of relevant statistics for the irregular kinematics. Once we have a validated model for the irregular kinematics and the associated sea surface, we can apply this in connection with our existing load models. As a benefit of focusing on the kinematics we also avoid relying on the often somewhat problematic scaling of lab loads.

However, we have a challenge that we cannot do bulk measurements of kinematics in a large volume of water in the middle of the 3D basin. Therefore, how do we generate relevant statistics for the kinematics in extreme irregular events for all sea-states tested?

The solution was to test "instrumented response models" directly in the basin. These response models should be the simplest possible structural models which assemble loads in the volume of water which will drive the failure mechanisms of interest. Keeping the structures simple, and by establishing an accurate link between lab-scale kinematics and lab-scale loads on the model, it is possible to isolate the statistical variation driven by the irregular kinematics. In other words, we use the normalized distribution (variation) of the loading on the tested response models and not the absolute value. This largely makes us independent of load scale effects as long as the load process in lab-scale is a reasonable approximation of the full-scale process. This is equivalent to conclusions reached by Tromans et al. [4] where they realized that normalized global loads on a large variety of offshore structures could be surprisingly well approximated with the global load (response) generated on a single vertical 1m diameter stick. They also used the hazard curve as the normalized way to compare the statistics across platforms and regional areas. Here we just use an extension of this principle. Instead of doing a theoretical estimate of the load on the response model ("the vertical stick") as done in reference 4, we take the response model into the wave lab and establish the load distribution by testing instead of performing theoretical calculations. As we are concerned that normalized statistics for mudline loads alone are not representative for our problem, we do not only test a vertical column for mudline loads, we test a column split at MSL. This allows us to do relative comparisons of the normalized load distributions on the lower and the upper section, respectively. In addition we test a vertical column with an air gap of 11m, representing a response model which is only affected by upper crest kinematics. Most importantly for our problem, we also test a generic deck structure to derive normalized load statistics for an open deck beam structure. By dividing the generic deck (spanning 40x40m) into 8x8 sections with individual load measurements at each section, we add the opportunity to evaluate differences in normalized deck response as a function of deck area size. Thus in Series "C" we make a rerun of all Series "A" sea-state tests with the following response models within the target area:

- "Generic Deck": Open deck structure. Sharp corner bar elements, 40x40x1m deck, air gap between 12 and 14m. 8x8 configuration with individual load monitoring on each 5x5x1m cross section (64 elements in the deck).
- "Drag" column: Vertical ø2m very rough column split at MSL. Individual load monitoring on each section.
- "WID" column: As per the Drag column, except mounted with an air gap of 11m.

A snapshot from a super slow motion video from the "C" series is shown in Figure 4.

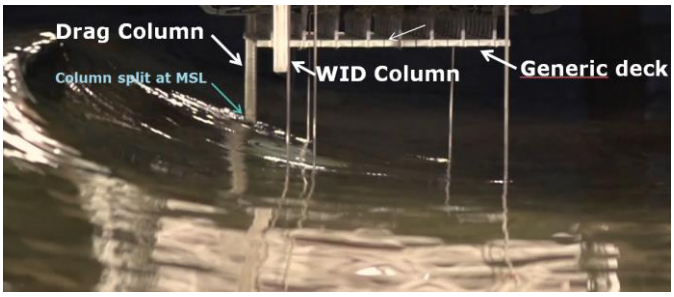

Figure 4. Response models tested in Series "C".

For the response model test strategy to work, it is key that we have a very detailed knowledge about the lab-scale load models for each of the tested models. Very controlled flume (2D) tests were applied to develop these models, as the glass wall allows direct measurements of the kinematics. The range of wave events which occurs in the basin (non-breaking, spilling breaking and plunging) was repeated in the flume to establish the kinematics. Repeating the test in the flume including the response models, the loads on each model were measured, thus enabling the lab-scale load vs kinematics model to be established.

Test series "K", "L", "M", "N" and "O" cover development and documentation of these flume tests targeting load model development. It covered 4 typical wave types:

- Wave II: Regular "Stokes 5th-like"
- Wave III: Irregular non-breaking (but slightly top-spilling, revealed in super slow-motion video))
- Wave IV: Violently spilling breaker
- Wave V: Plunging breaker

These waves were investigated in detail by Particle Image Velocimetry (PIV) and super slow motion video ("K" series).

Test series "A", "C" and the flume load model series are the core of the lab test work performed. In addition a smaller set of tests were performed to study details. A few of these are:

- Series "B": 18 selected extreme irregular wave events picked from the "A" series. 3 long crested plungers, 3 short crested plungers; similar 2x3 spilling breakers and 2x3 non-breaking waves. Short time series around each event tested and repeated 10 times to estimate repeatability. The main purpose for this test is to allow CFD validation, generating the events in CFD by direct transfer of wave paddle motion as boundary conditions to the CFD model.
- Rerun Series "B" with the response models in the target area.
- Series "P": A number of unidirectional sea-state tests performed in the flume (H_s, T_p combinations).
- A temporary wind tunnel was built above the flume to allow a small study of wind/sea interaction repeating a few of the "P" series tests with a wind field above the surface. PIV was used to map the wind induced current (and associated wind driven wave) field.
- Detailed flume study of wave gauge performance in very steep waves (applying Wave V from the Series "K").

In addition to the core test work performed by DHI a detailed assurance program was performed at Imperial College (IC). The main purpose of this program was to provide independent assurance of the DHI work. The IC program included:

- Validation of DHI "A" series tests by running a selected range of core sea-states in the IC lab (IC lab has been investigated in detail and validated as part of the CresT [10] and ShorTCresT [11] JIP programs).
- A very detailed study of crest kinematics in focused breaking waves using LDA.
- Performance of an independent "Standard Industry WID test" at scale 1:72. For this test IC built a detailed model of the Tyra West C wellhead platform and tested this in selected extreme sea-states. This test was used to validate the WID simulation approach applied in the Monte Carlo simulation, see reference 1 for further details.

5 TEST RESULTS

5.1 A Series, Crest Height and Breaking Probability

A few of the result plots provided by the ePlot application are sketched in Figures 5-6.

Figure 5 is taken as an example of a very steep sea-state dominated by highly non-linear events. Throughout the "A" tests we see the same trends. For the steepest events, amplifications relative to a Forristall distribution are typically seen. Recalling that the Forristall distribution is to 2nd order, this is concluded to be higher than 2nd order effects. Ultimately in the steepest sea-states extensive breaking starts to limit the crest height and the test data curves back and sometimes inside the Forristall distribution again. It is noticeable that even in the steepest sea-states highly influenced by breaking, we never really saw evidence of the existence of a limiting crest height introduced by breaking. As seen in Figure 5 there is still a significant gap between the point statistics and the area maximum statistics. If we were approaching a limiting value, these two distributions should converge. In relation to crest statistics it is noted that the IC assurance tests showed excellent agreement with the DHI "A" Series tests.

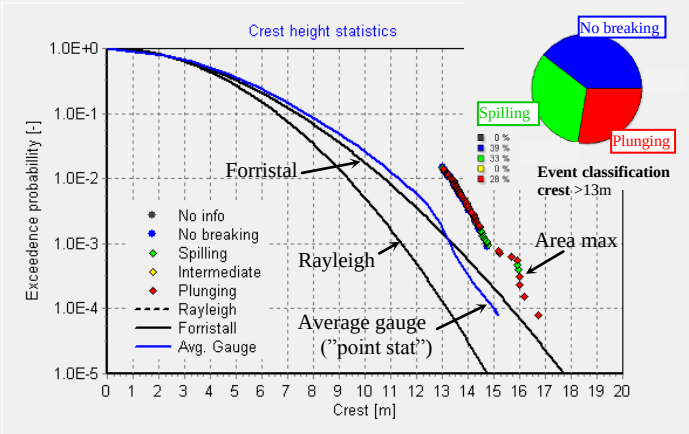

Figure 5. Plot of crest height exceedance probability and distribution of "wave type" for crests exceeding 13m in the 72x72m target area. H_s=12m, T_p=12.5s and s_p=20.

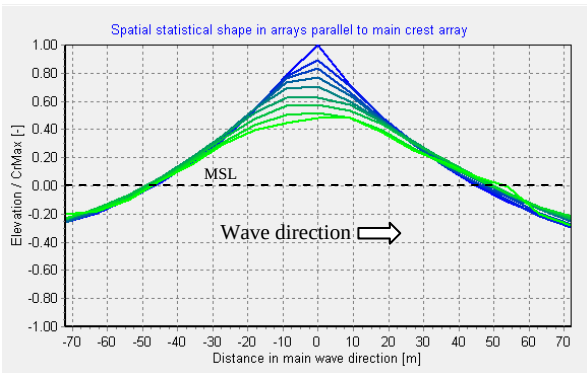

Figure 6. Avg. normalized shape contours of waves at peak crest height for crest height > 13m in target area. Contours at 9m spacing. H_s=12m, T_p=12.5s and s_p=20.

ePlot has a number of plot features aimed at presenting the massive amount of test data in a simple directly accessible way. One feature is to plot the average normalized shape of the classified events (crest>13m, at least the 50 largest events in each test, more than 7000 classified in total). This plot is established using the data record from the grid of wave gauges. An example is shown in Figure 6. Recalling the discussion on importance of logging frequency, and now knowing the average crest shape and that the celerity of the events classified in the plot is between 16 and 21 m/sec, we can evaluate the logging error from Figure 6. The average crest is "sharp edged" and it is seen that on average a 2Hz logging will under-predict the crest height by approximately 5% for this test (at 2Hz a measurement is taken every 8-10m) .

Based on video recordings of the 7000 classified events we can plot the breaking probability (crest height>13m, at least 50 events/test) on the environmental contour, see Figure 7.

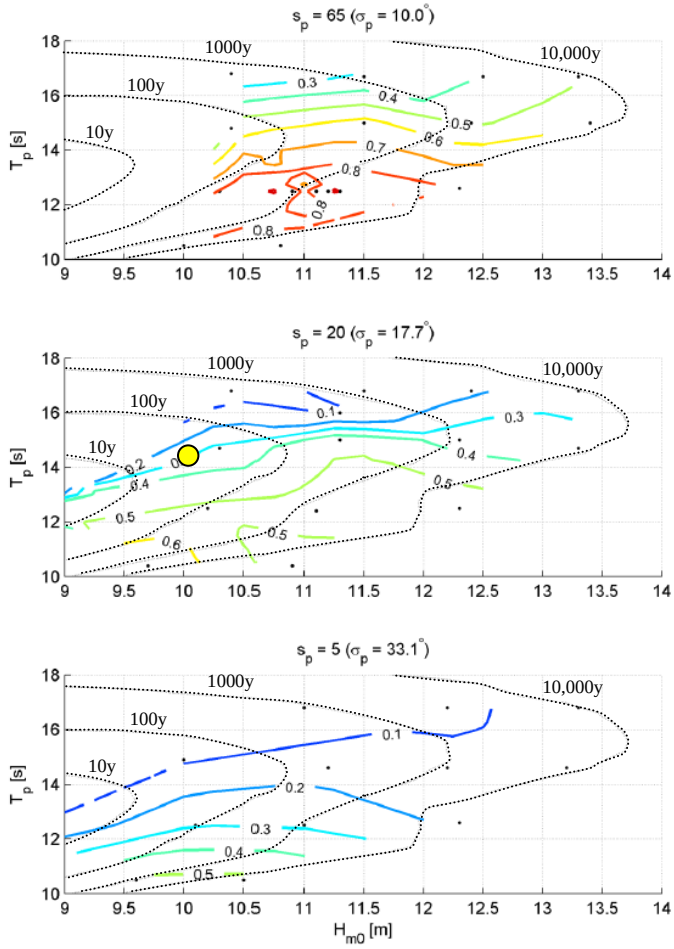

Figure 7. Breaking probability for crest height >13m (min. 50 largest crests/test) as function of H_s, T_p and s_p plotted on top of the environmental contour. Yellow dot: Tyra storm w. breaking observed. Black dots: Sea-states tested.

In the 10m H_s storm conditions where breaking waves were observed in the field, there was approximately a 30% probability that a large crest would belong to a wave which would break in or near the observation area. On average, of the 7000 events classified, every 3rd to 2nd event was breaking. A further conclusion from Figure 7 is the relatively large influence of directional spreading. For constant H_s and T_p the breaking probability is largest in the less spread sea-states. The reason for this is likely the less "effective" steepness of directional spread events caused by the vector summation of the directional components.

PIV measurements were successfully completed in the flume for Wave II-V. The regular (II) and non-breaking (slightly top-spilling) (III) waves revealed crest velocities as expected around 10 m/sec. The velocity increased in the spilling breaker (IV) and in the plunging breaker (V) up to 20-25 m/sec. was measured, see Figure 8. As part of the study substantial effort was put into CFD (volume of fluid) analysis, the main goal being to act as an assisting tool to provide a better understanding of the kinematics in the 3D basin. The CFD approach was based on directly applying the wave maker motion as boundary conditions to the analyses, i.e. true non-linear boundary conditions and no approximations.

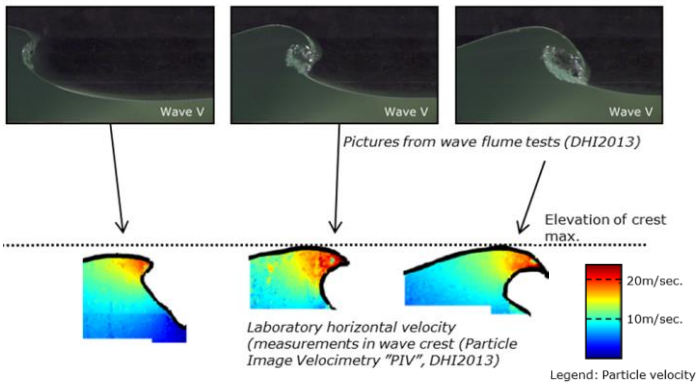

Figure 8. Snapshot and PIV results of Wave V.

The STAR CCM+ was the code found to perform best. The optimal setting, mesh, solver, etc. were determined for the 4 flume waves (Wave II-V) using the PIV as reference. Very good performance of the CFD was found. Once having a validated approach in the flume, the same set-up was used to analyse the 18 selected 3D sea-state basin events ("B"-series). This data was used as inspiration for development of the semi-empirical sea-state model. Even though these events are purely a spot check of selections from the "A" series, they give a clear indication that at the point of overturning, crest kinematics are very similar in 2D and 3D as the range of loads on the upper section is identical.

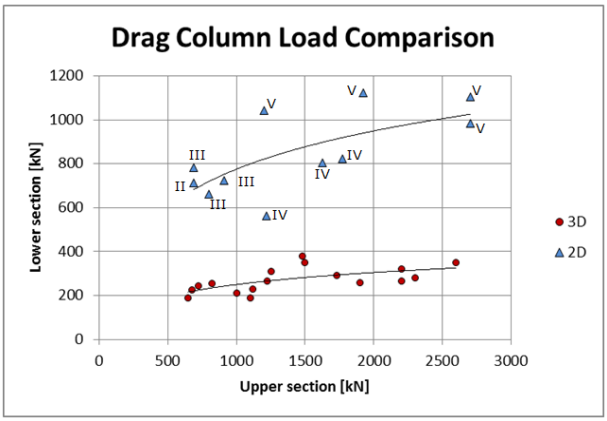

Figure 9. Comparison of load measurements on the Drag column upper and lower sections. Red dots: 3D irregular extreme events. Blue triangles: Flume 2D tests (Wave II-V). Note, in the flume the column was tested at different positions along the flume for Wave III-V.

This is in agreement with the visual (super slow motion video) observation that once a 3D wave overturns, the surface is close to 2D, which is probably due to alignment of the higher frequencies taking part in the overturning process. This could indicate that the "kinematics reduction factor" commonly applied within industry to reduce the 2D regular wave kinematics may not be a constant as normally assumed:

it will likely be a more complex function of the non-linearity and dependent upon the elevation in the water column.

If we take a quick look at the loads recorded at the lower and upper section of the Drag column in the flume 2D test events and the 18 3D test events, Figure 9, we see a clear indication that from MSL and up the 2D and 3D results are very similar. However, in these strongly irregular events there are much lower loads from MSL and down.

6 ASSURANCE

Being a part of the Tyra "As-Is" Safety project, the work presented in the present paper has been subject to external assurance in addition to the "in project" assurance already listed (independent testing, CFD, etc.). External assurance covered technical expert reviews by DUC partner Shell (K. Ewans and T. Rhodes), two independent second party reviews by KBR/Energo and Dr. Graham Stewart, respectively, and a complete 3rd party project verification by DNV (DNV Denmark assisted by experts from the headquarter in Høvik).

7 CONCLUSION

In a need to explain the occurrence of two extreme plunging breakers in the Tyra field, we have performed a large systematic wave basin (physical and numerical) test program. The main target of the wave study was to clarify and understand issues related to loading generated in extreme irregular sea-states and as part of this discuss and understand differences to currently adopted industry approaches/ assumptions. The main conclusions from this "New Knowledge" are:

- The nature of extreme non-linear irregular events (ultimately breaking) is conflicting with many of the assumptions underlying current industry practice:
  a) Effects greater than 2nd order can drive higher point crest statistics. Compared to using the standard Forristall statistics, we found a 6% increase in the 10,000-year return period crest height for Tyra.
  b) Kinematics in breaking waves are transient and very high crest velocities can occur. In severe plunging events (for d=45m) peak crest particle kinematics may be of the order of 20-30% higher than the celerity.
  c) We see indications that the 3D "kinematics factor" effect is not constant over the height of the water column. Close to 2D conditions occur in the crest while 3D effects are apparent deep in the water column.
- For the assessed case, Southern/Central North Sea, d=45m, the non-linear extreme events dominate the extreme waves within the 10-10,000y environmental contours. Looking at tests results for sea-states between H_s=10m and 13m we find that approximately every 2nd or 3rd event with crest height >13m belongs to a wave which is breaking in the vicinity of the measurement point.
- In highly non-linear irregular wave events, due to the transient nature of the kinematics, there is not a unique link between wave (or crest) height and wave load (on a given structure). Different events (or different "timing" of the same event) may lead to large variations in loads down the water column. It follows that variations in loads acting on each potential failure mechanisms will result.
- Irregular wave loading will, due to the above findings, challenge the often applied assumption of a deterministic link between global loads and global failure. Many different collapse mechanisms in a structure can lead to global collapse, and, except for the foundation, these mechanisms are commonly driven only by a subset of the global loads. To deliver wave loads as input to SRA it is important to ensure consistency between load statistics and structural failure mechanisms.

The companion paper [1] provides a further discussion of the results and their impact in relation to SRA.

ACKNOWLEDGEMENTS

The present paper is based on work performed by the Tyra "As-Is" Safety Project team members, who all took on a huge task to execute the project on a much challenging time schedule. Other key team members working on the environmental models are Hans Fabricius Hansen (DHI); Niels-Erik Ottesen Hansen and Francesco Stevanato (LIC Engineering); Björn Ullbrand (Validus Engineering AB) and Prof. Chris Swan (Imperial College). It is expected that the project team in the future will release more in-depth topic specific papers to supplement the summary in the present paper and the companion paper [1].

Furthermore, thanks to the DUC partners (Shell, Chevron and Nordsøfonden) for active contribution in technical workshops, etc.

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[11] ShorTCresT JIP, *Effect of Short-crestedness on Extreme Wave Impact*, joint industry project managed by Marin, 2011-13

Chapter 4

Session 2: Wave Environment and Loads

4.1 Presentation by Guido Kuiper

NTNU Norwegian University of Science and Technology

International Association of Oil & Gas Producers

Demonstrating Draugen NORSOK compliance despite significant wave load increases since original design

Guido Kuiper
A/S Norske Shell

The 3rd Offshore Structural Reliability Conference
OSRC2016
14-16 September, Stavanger, Norway

Introduction to Draugen

Draugen Platform Facts

- Oil field in the Norwegian Sea, ~150 km north of Kristiansund
- Concrete mono-column structure carrying a 22.000 t topsides
- Located in 250m water depth
- Subjected to severe weather conditions

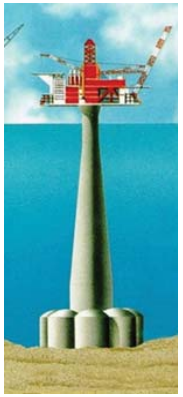
Introduction to Draugen

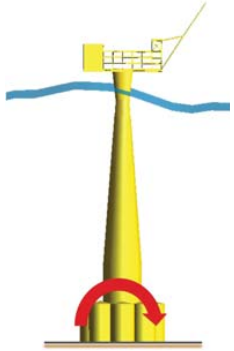
Draugen History

- Started production in 1993
- Field had an estimated life of 20 years and an estimated recovery rate of under 40%
- Currently one of the Norwegian fields with the highest recovery rate of around 70%
- Production licence ends in 2024
- Draugen was built in accordance with the Norwegian regulations

Draugen Structural Integrity

- Over Turning Moment at the base of the shaft is the critical structural response
- Special attention to 'ringing' phenomenon





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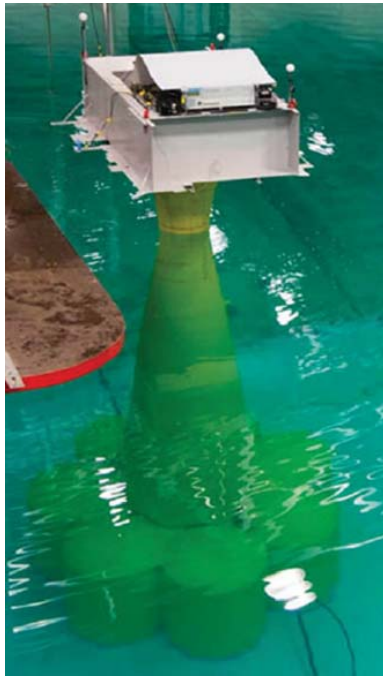
Marintek Model Test Campaign

- 1:50 scale model
- **Play movie**



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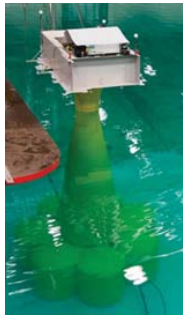
Limit State Design

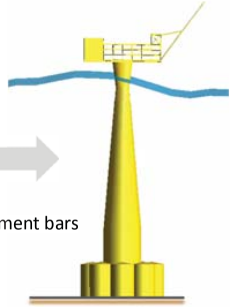
- Accidental Limit State (10.000-yr check) ✓

10.000 year response for linear elastic model



Cracking of concrete
Yielding of reinforcement bars

10.000 year response for non-linear elasto-plastic model



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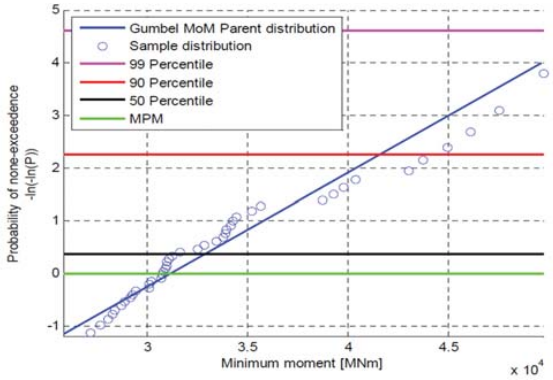
Marintek Model Test Campaign



- Model test responses of 1992 and 2014 campaigns are in the same range
- Due to NORSOK requirements, characteristic wave loads <u>increased significantly</u> (~30%) by applying 90% level instead of MPM

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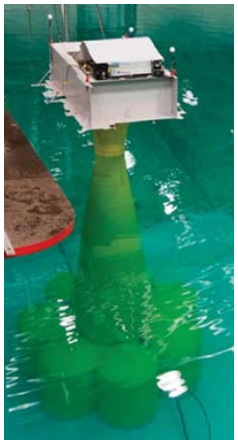
Intermediate Conclusion

- Accidental Limit State (10.000 yr) – Compliant with NORSOK.
- Ultimate Limit State (100 yr) – Not compliant with NORSOK.
- Compliant with Shell Design and Engineering Practice (DEP) on fixed concrete structures.
- Shell assessed the GBS to have an acceptable structural integrity.

Additional work

1. Wave Spreading
2. Long Term Analysis
3. ULS check
4. GBS Capacity



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GBS Life Time Extension – Main Contributors

- Shell
  - A/S Norske Shell
  - Shell International E&P
  - Shell Safety Reps.
- Kværner (Leading Contractor
  - Marintek, Imperial College (model test)
  - Dr. Techn Olav Olsen (GBS capacity)
  - Aker Solutions (LTA)
  - NGI (soil)
- Reinertsen (third party verification)
  - Sintef, Imperial College, others
- PSA

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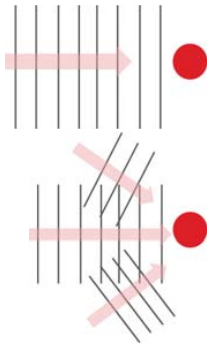
1. Wave Spreading

Knowledge based on Short Crest JIP and recommended by 3rd party verification team.



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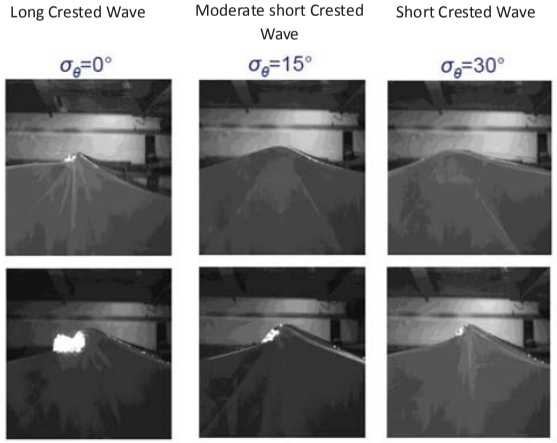
1. Wave Spreading



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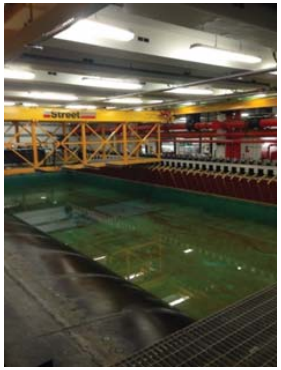
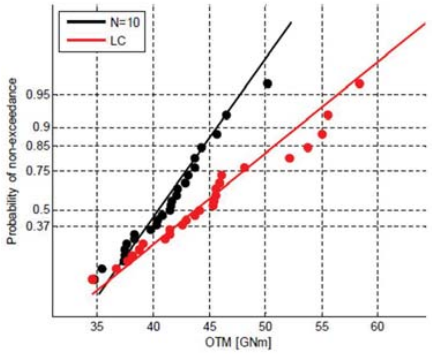
1. Wave Spreading

Wave spreading has two effects

- Direct – Reduction in wave velocity, wave acceleration and wave steepness
- Indirect - Reduction in crest elevation for a given exceedence probability



Wave Basin at Imperial College



- Impact from wave spreading, <u>12% reduction in overall platform response</u>

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2. Long Term Analysis

Key question

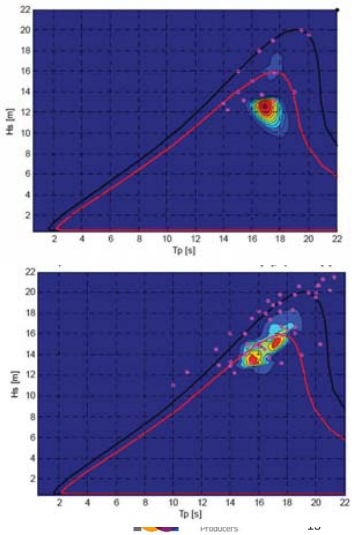
- What is the true 100-year Draugen response rather than the assumed 100-year response?
  - Limited data from Marintek model test campaign (number of sea states, data points within a sea state)

Improvements

- Obtained wave basin data from Statoil (now 800 individual tests)
- Conducted additional 10-year sea states to improve statistics

Impact

- Draugen extreme response reduced by 8% to 14%



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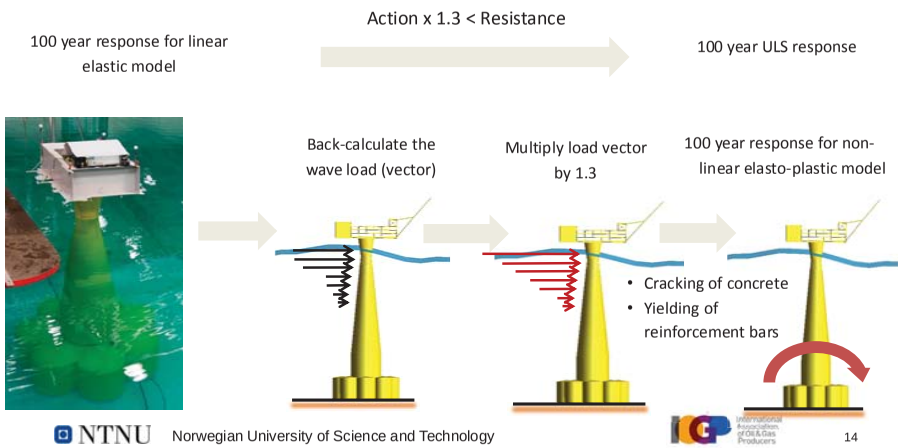
3. Ultimate Limit State (ULS) Check



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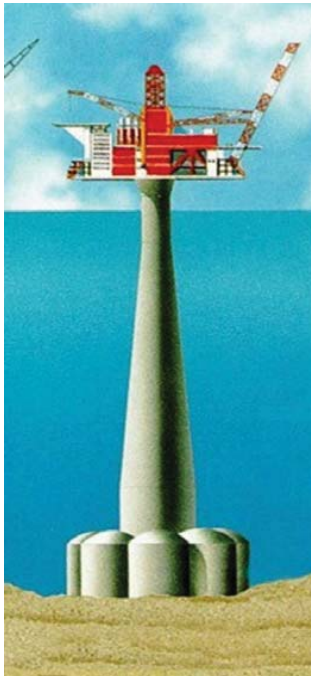
4. GBS Capacity

- Dewatering of 5m increases structural capacity of GBS shaft by 5%-6%
- Dewatering to a lower water level (5m-30m) will increase structural capacity of GBS shaft but will not increase overall capacity of the platform



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Final Conclusions

- Draugen continues to have satisfactory structural integrity.
- ULS and ALS in compliance with NORSOK.
- **PSA has granted Approval for Life Time Extension till 2024 (end of production licence).**
- Draugen to be considered as future hub in Haltenbank area.

Outcome only possible due to excellent Team Work with contractor, sub-contractors, PSA and 3rd party review team!

Questions and Answers

4.2 Presentation by Gunnar Lian

NTNU Norwegian University of Science and Technology

International Association of Oil & Gas Producers

Slamming Loads from Steep and Breaking Waves

Gunnar Lian/Statoil,Norway
Tone Vestbøstad/Statoil,Norway

The 3rd Offshore Structural Reliability Conference
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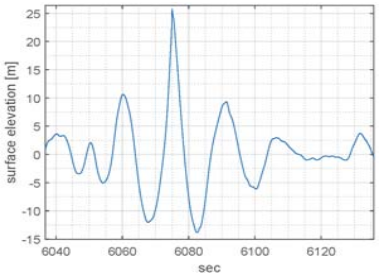
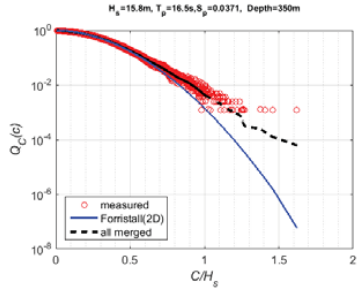
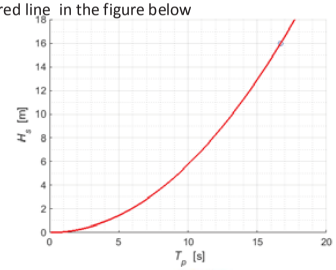
Steep breaking waves





Deep water →scale by steepness-→ The distribution in the upper figure is valid for all sea states along the red line in the figure below

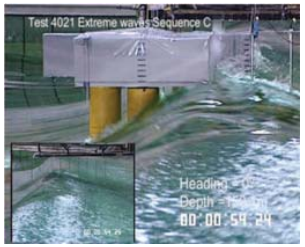


Introduction

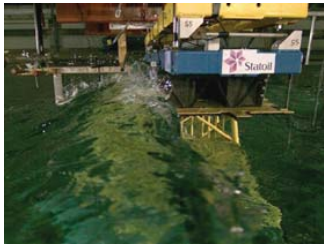
- How can we estimate the q-annual design value for slamming ?
  - The correct approach will be to do a full long term analysis
- OK, is that straight forward ?
  - Well, first you need to establish the short term distribution of the slamming loads in a sufficient number of sea states.
- Ok, is that straight forward ?
  - Well, it is almost impossible to do that numerically, and definitely impossible to measure in full scale. So the only option available will be to do a small scale test, but still you need A LOT of tests.
- Hmm, are there any way we can reduce the number of model tests?
  - Well, a simplified approach is to identify the worst q-annual sea state, and then use an appropriate percentile for the short term distribution. The appropriate percentile should ideally be estimated from a long term analysis. For slamming column slamming, the percentile has previously been estimated to be extremely high (>99%).
- Ok. Now that we have learned how to do it correctly. We will simplify in this presentation, and use the 90th percentile.

Introduction cont

- Wave in deck impacts have been a research topic for several years. *



GBS –2002. Photo:Marintek



Jacket –2011. Photo:Marin

Highest measured load ~ 1.75MN/m

Scharnke, J., et al. (2014). Wave-in-deck impact load measurements on a fixed platform deck. Proceedings of the International Conference on Offshore Mechanics and Arctic Engineering - OMAE

* a review of the topic and the use CFD can be found in :Vestbøstad, T. M. (2009). A Numerical Study of Wave-in-Deck Impact using a Two-dimensional Constrained Interpolation Profile Method. Department of Marine Technology, Norwegian University of Science and Technology. **philosophiae doctor**

Column slamming

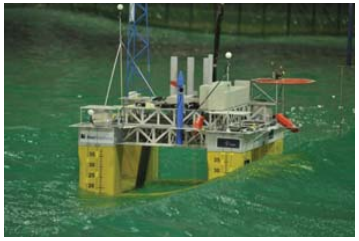
- The topic was brought to attention by Dr. Sverre Haver and Dr. Chris Swan during a model test at Imperial were they reported slamming pressure of the excess of 3MPa, on a significantly area.

Based on the test results, Dr. Sverre Haver recommended to use 3MPa on 50 m^2, if not documented otherwise.

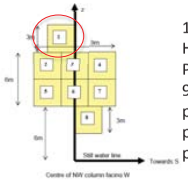
GBS –2007. Photo from ref:

Roos, J., et al. (2010). Wave impacts on the column of a gravity based structure. Proceedings of the International Conference on Offshore Mechanics and Arctic Engineering - OMAE2010, American Society of Mechanical Engineers

Column slamming

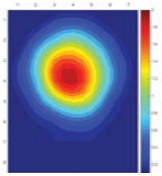


Midgard –2010. Photo:Marintek



18 realizations
Hs=20.2m Tp = 21s
Panel1,
9m2
p90 = 2.3MPa
p95 = 3.0 MPa
p99 = 4.4 MPa

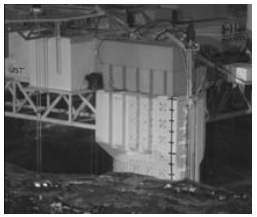
Heidrun –2013. Performed 300 x 3hour tests

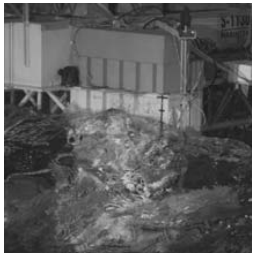


113 realizations
rate of slam H_s = 18.2m and T_p = 16s.
9m2,row4: p90 = 2.4MPa,
p95 = 2.9 MPa
p99 = 4.2 MPa

42 panels

Column and deck slamming on production Semi





~ 350 x 3-hour

Incident that brought the attention to deck-box slamming for the Mobile Units (MOUs).

Figure 3 Forward bulkhead of the CI berthed at CCB after the incident. The scale of the damage can be seen from the broken windows and fastening brackets for cable trays.

from PSA Investigation report

http://www.psa.no/investigations/investigation-report-into-a-fatal-accident-on-coslinnovator-on-30-december-2015-article12005-893.html

http://www.psa.no/enforcement-notices/amendment-of-decision-withdrawal-of-order-for-cosl-drilling-article12206-892.html

https://www.dnvgl.com/news/damage-to-topside-structure-of-column-stabilized-units-due-to-direct-wave-impact-66805

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Deck box slamming

- Statoil has this summer performed modeltest on two MOUs to estimate slamming loads on deck-box
- Airgaps were 14.5 m and 16.6 m
- We will present normalized results for SEMI-Cat D with 16.6 m airgap

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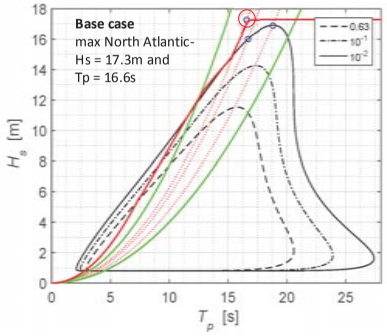
ULS test cases

Case	Hs[m]	Tp[s]	Spectr.	C[m/s]	W[m/s]	D[deg]
1	17.3	16.6	JONS	1.0	32.5	180
2	17.3	16.6	JONS	0	0	180
3	17.3	16.6	JONS	1.0	32.5	165
4	16.0	16.7	JONS	1.0	32.5	180
5	16.8	18.8	Torseth	1.0	32.5	180

All tests are with long crested waves



4x10 Grid of force panels

Cat D

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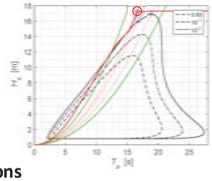
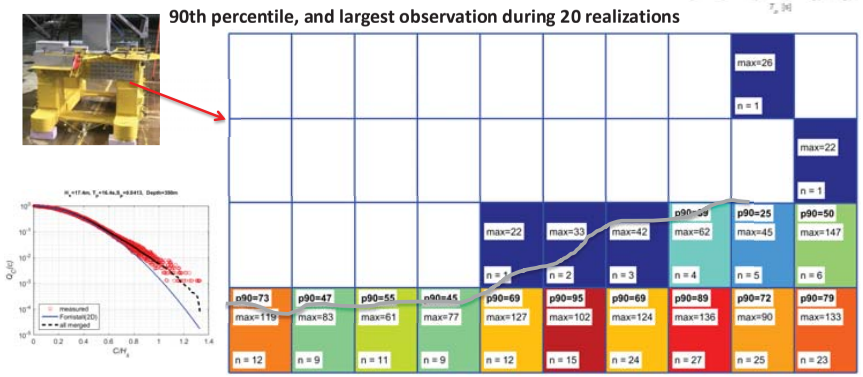
Max North Atlantic- Hs = 17.3m and Tp = 16.6s, head on, including wind and current.



90th percentile, and largest observation during 20 realizations



n = total observation above threshold during 20 realizations

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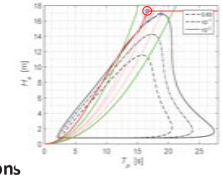
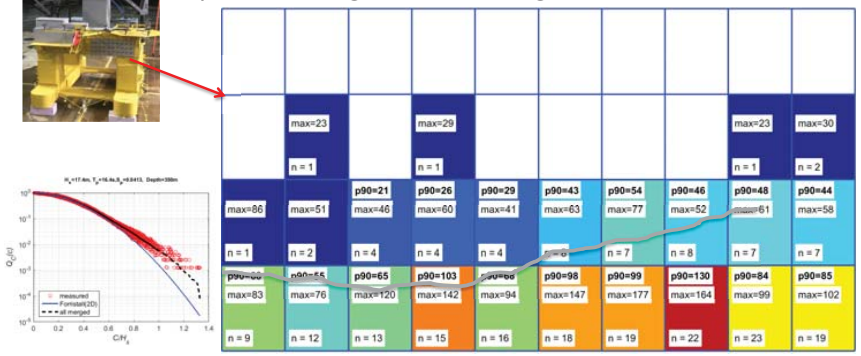
Max North Atlantic- Hs = 17.3m and Tp = 16.6s, 15 degree of. Including wind and current.



90th percentile, and largest observation during 20 realizations



n = total observation above threshold during 20 realizations

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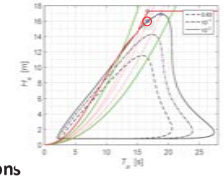
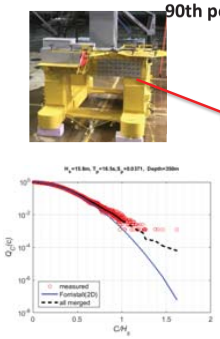
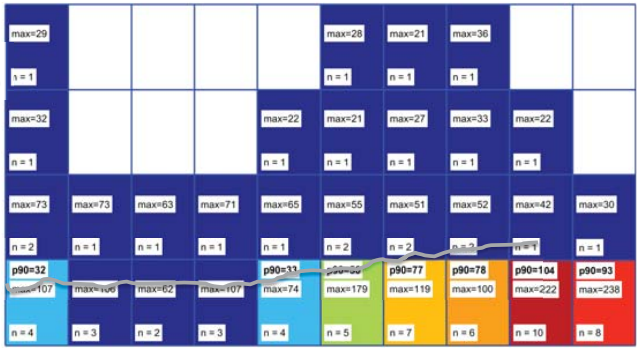
Hs = 16.0m and Tp = 16.7s,
Head on. Including wind and current.



90th percentile, and largest observation during 20 realizations





n = total observation above threshold during 20 realizations

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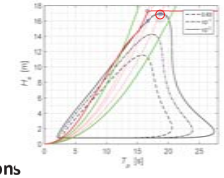
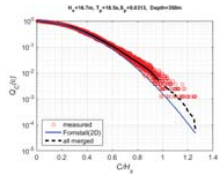
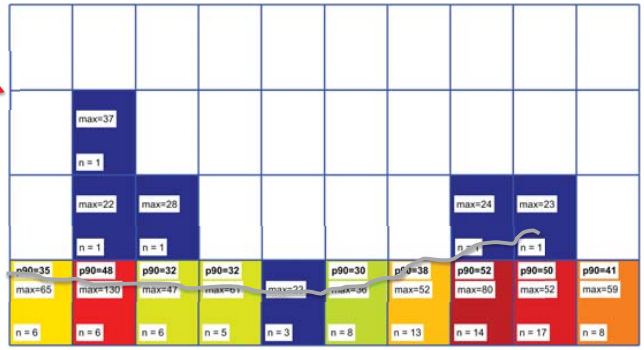
Hs = 16.9m and Tp = 18.8s,
Head on. Including wind and current.



90th percentile, and largest observation during 20 realizations





n = total observation above threshold during 20 realizations

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Summary MOU tests

Case	Hs[m]	Tp[s]	Spectr.	C[m/s]	W[m/s]	D[deg]	Max p90
1	17.3	16.6	JONS	1.0	32.5	180	95
2	17.3	16.6	JONS	0	0	180	67
3	17.3	16.6	JONS	1.0	32.5	165	130
4	16.0	16.7	JONS	1.0	32.5	180	104
5	16.8	18.8	Torseth	1.0	32.5	180	52

- A priori, head seas was assumed to give largest design slamming loads, but 15 degrees off head seas gave similar or worse results
- Due to the large uncertainties, some conservatism should be included to account for not measuring at the worst area on deck box, worst sea state, worst direction etc.

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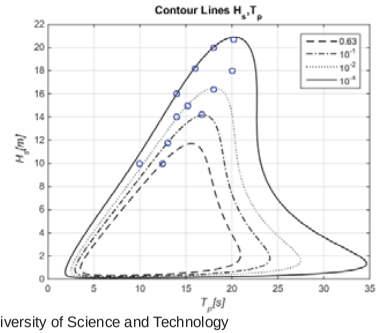
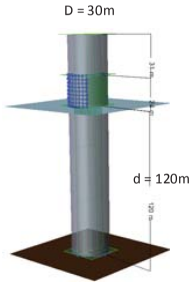
Very near future research

Estimate the effect of wave spreading of cos n=10 on

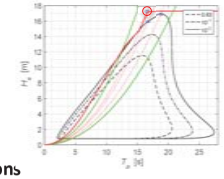
The Research will be performed at Imperial College, supervised by Professor Chris Swan

Annual prob.	H_s [m]	T_p [s]	No of seeds
0.63	10	12.5	80
10^{-1}	11.8	13	60
10^{-1}	14	14	60
10^{-2}	14.2	16.7	60
10^{-2}	15	15.2	60
10^{-2}	16	14	60
10^{-3}	10	10	60
10^{-4}	16.4	18	60
10^{-4}	18	20	60
10^{-4}	18.2	16	60
10^{-4}	20	18	60
10^{-4}	20.7	20.2	60
		Total	740 x2



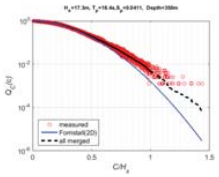


Max North Atlantic- Hs = 17.3m and Tp = 16.6s, head on, without current and wind.



90th percentile, and largest observation during 20 realizations

max=47 n = 1	max=120 n = 1								
max=92 n = 1	max=129 n = 1	max=112 n = 1							
max=246 n = 1	max=313 n = 1	max=208 n = 1	max=118 n = 1	max=97 n = 1	max=45 n = 1	max=39 n = 1	max=34 n = 1	max=29 n = 1	
p90=67 max=150 n = 7	p90=45 max=113 n = 4	p90=53 max=95 n = 4	p90=36 max=84 n = 5	p90=37 max=89 n = 7	p90=43 max=100 n = 5	p90=63 max=135 n = 11	p90=63 max=126 n = 13	p90=56 max=93 n = 10	p90=41 max=62 n = 5



n = total observation above threshold during 20 realizations

Distribution of slamming loads

The exact distribution of the extremes in a sample size of n-independent observations is given by:

$$F_{X_{3h}}(x) = (F_X(x))^n \quad (1)$$

The number of slams n in each 3-hour is also a statistical variable. If we assume this to be Poisson distributed, the exact distribution is:

$$F_{X_{3h}}(x) = \sum_{n=0}^{\infty} F_X{}^n(x)\exp(-\lambda)\,\frac{\lambda^n}{n!} = \exp\big(-\lambda(1-F_X)\big) \quad (2)$$

Initial distribution of an exponential type lead to Gumbel distribution, for increasing n (Bury, 1975) :

$$F_{X3h|HpTp}(x|h_s,t_p) = \exp\left\{-\exp\left[-\left(\frac{x-\alpha(h_s,t_p)}{\beta(h_s,t_p)}\right)\right]\right\} \quad (3)$$

We have use eq.2 and assume a 3-parameter Weibull initial distribution.
Note that the estimated extreme can be sensitive to the selected threshold.

Chapter 5

Session 3: Reliability of Jackets in Severe Wave Conditions

5.1 Presentation & article by Jesper Tychsen, Søren Risvig, Hans Fabricius Hansen, Niels-Erik Ottesen Hansen & Francesco Stevanato

NTNU Norwegian University of Science and Technology

International Association of Oil & Gas Producers

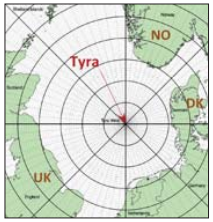
Summary of the impact on structural reliability of the findings of the Tyra Field Extreme Wave Study 2013-15

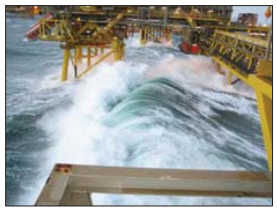
Jesper Tychsen, Structural Engineering Advisor
Maersk Oil, Danish Business Unit

The 3rd Offshore Structural Reliability Conference
OSRC2016
14-16 September, Stavanger, Norway

Agenda:

- Introduction to study
- Monte Carlo based simulation model
  - Long-term environmental statistics
  - Semi-empirical sea state model
  - Response model
  - Simulation set-up
- Selected results
- Comparison to existing SRA
- Conclusion





Plunging breaker at Tyra West

Introduction:

- Discovery of site evidence of breaking extreme waves 2012
- Further concern due to CresT and ShorTCresT JIP findings
- Existing extreme wave design criteria challenged
- Tyra "As-Is" Safety project 2013-15
- Target: Unbiased estimate of the annual failure probability of Tyra structures due to "New Knowledge"
- Solution, to allow non-linearities, avoid bias and allow inclusion of multiple failure mechanisms and correlations we need:
  - Monte Carlo based simulation model (1 million life-cycles)
  - Semi-empirical sea state model
  - Multiple failure mechanism response model
- Very high focus on assurance
- Third Party certification complete autumn 2015

High-level MC simulation set-up:

- "Simu1":
  - Generate a file of 1h sea states (Hs;Tp; sp) which can generate a potential collapse situation (1e6 lifecycles)
  - Add stochastic variation to all associated parameters (environmental, strength, subsidence, model uncertainties, etc.)
- "Simu2":
  - Realisation of the "Simu1" sea states
  - Identify extreme events and store "time" and "seed"
- "Simu3-4":
  - Check for collapse in all failure mechanisms in structure ("load">"resistance") for all extreme events.
  - Calculate failure probability.

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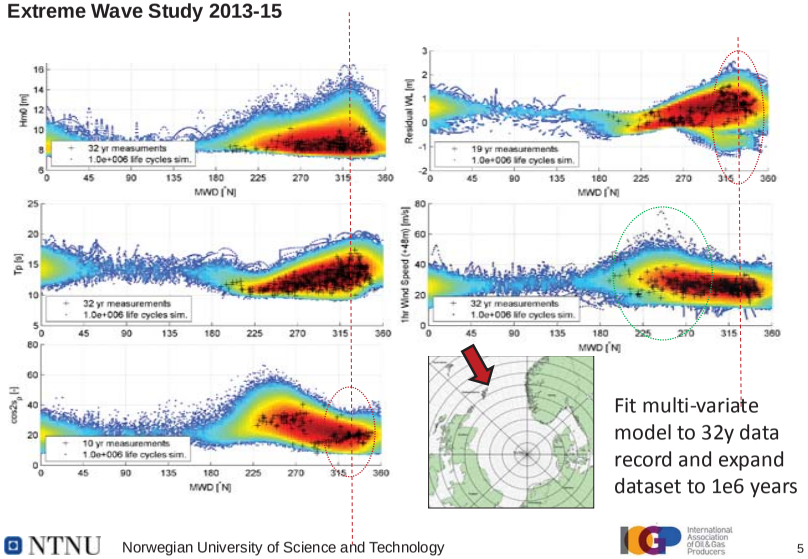
Summary of the impact on structural reliability of the findings of the Tyra Field Extreme Wave Study 2013-15



Fit multi-variate model to 32y data record and expand dataset to 1e6 years

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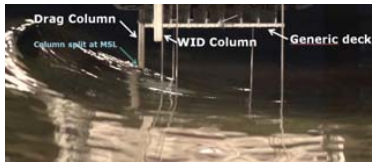
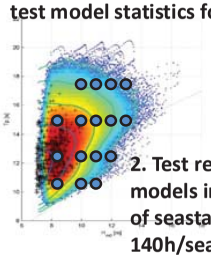
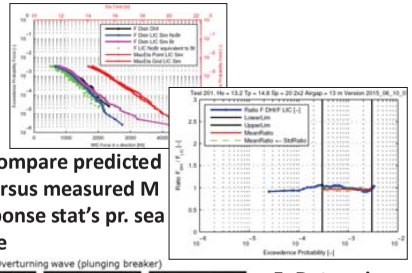
Semi-empirical sea state model development:



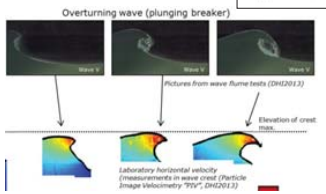
1. Use simplest possible "response" test model statistics for calibration



2. Test response models in a matrix of seastates (60-140h/seastate)



4. Compare predicted P versus measured M response stat's pr. sea state



3. Develop lab-scale load models for the response structures

5. Determine bias & COV on "P/M" (model uncertainty)

6. Loop over "4" and "5" to reduce COV

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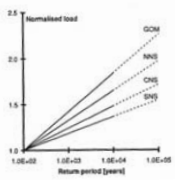
Response models:

- Normalised response statistics used by Shell in the Mid 90'ies to develop a general SRA model
- For mudline loads in regular waves:
  - In the same long-term (regional) environment normalised BS/OTM statistics are "identical" for a large range of structures
  - => Vertical column (OD=1m) used in Shell SRA
  - Model used to derive regional RSR values and estimate partial coefficients
- We need an extended response format to include all principal failure mechanisms to allow improved assessment of irregular wave loading



The hazard curve = normalised loads statistics

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- 1st model extension: Level Shear and OTM

- 2nd: Bar & Membrane (B&M) model:
  - Non-linear force flow during failure mechanism development
  - At system capacity loads are shed statically determinate between failure mechanisms
  - => Distribution of failure mechanism loads can be often be approximated by a simple system of Bars & Membranes
  - =>simple load mapping "influence factors" to individual failure mechanisms can be applied
  - The B&M model can:
    - Trace loads in all major failure mechanisms
    - Include load correlations between failure mechanisms
    - Combine "gravity with OTM" and "torsion with BS", e.g. offset WID loads will load one membrane face element more than the opposite
    - Model stochastics collapse of failure mechanism

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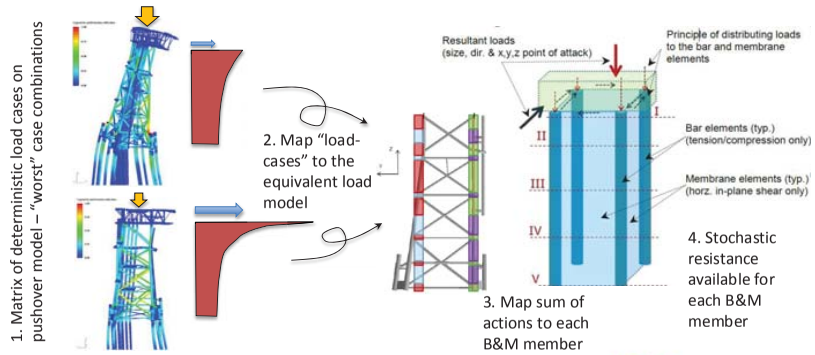
We prepare a B&M model for each structure:

- Calibrate a Morison "point" equivalent load model
- Calibrate resistance of each failure mechanism to the pushover model



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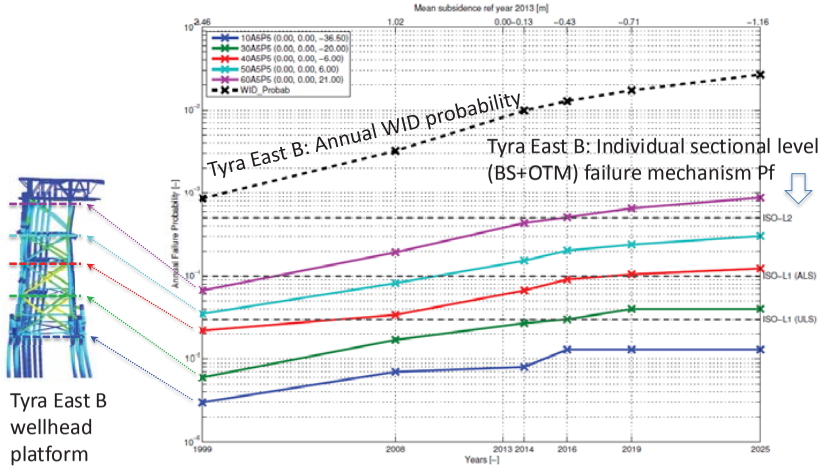

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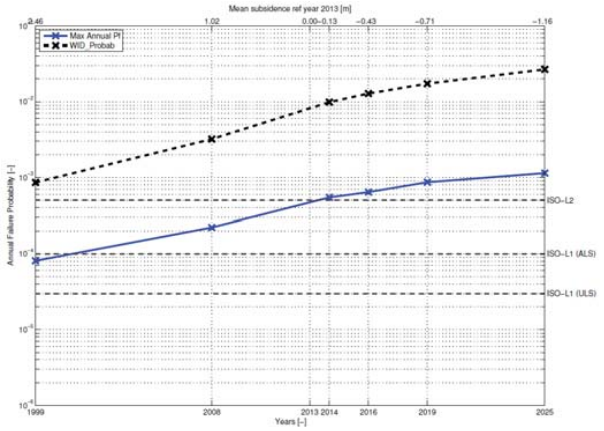
Tyra East B System annual Pf:



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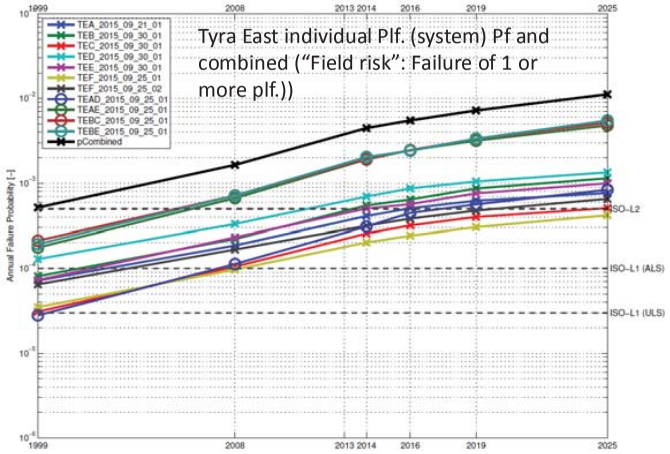
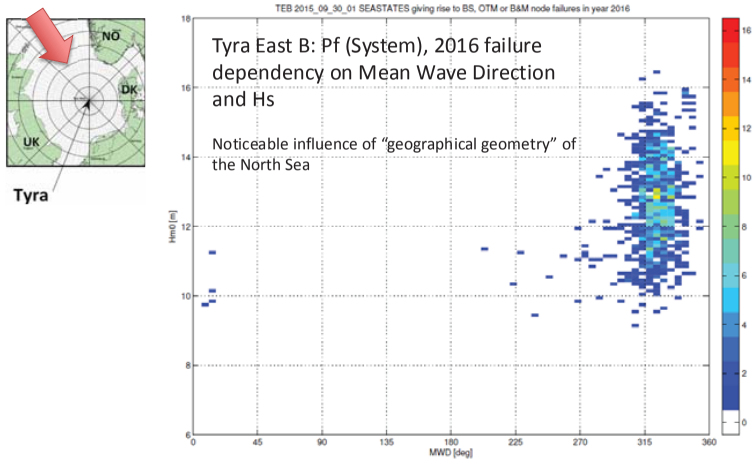
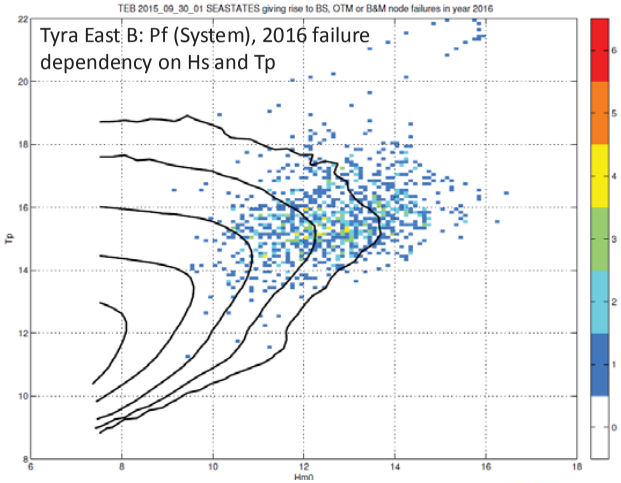
Summary of the impact on structural reliability of the findings of the Tyra Field Extreme Wave Study 2013-15



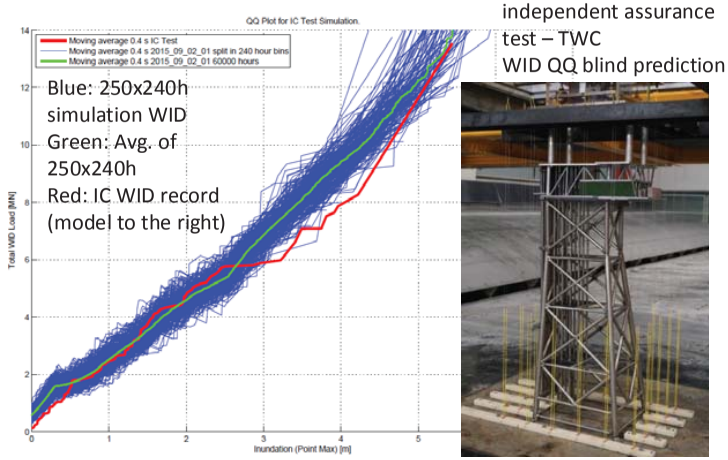
Summary of the impact on structural reliability of the findings of the Tyra Field Extreme Wave Study 2013-15



Summary of the impact on structural reliability of the findings of the Tyra Field Extreme Wave Study 2013-15



Summary of the impact on structural reliability of the findings of the Tyra Field Extreme Wave Study 2013-15

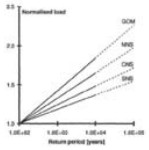


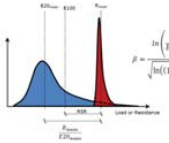
#16

Summary of the impact on structural reliability of the findings of the Tyra Field Extreme Wave Study 2013-15

Shell 1990'ies SRA model:

- Statistical model using "response" (load sum) of environmental loads
- Global base shear on 1m vertical column used
- Approach:
  - Hindcast database (NESS≈25y) of wind, wave, etc.
  - Most probable load (Emp) and linear short term CoV
  - Regular (or Newwave) wave theory
  - Exponential Distribution (log linear hazard curve)
  - SDOF reliability model
  - Log-Normal distributions
  - Results used to access all failure modes





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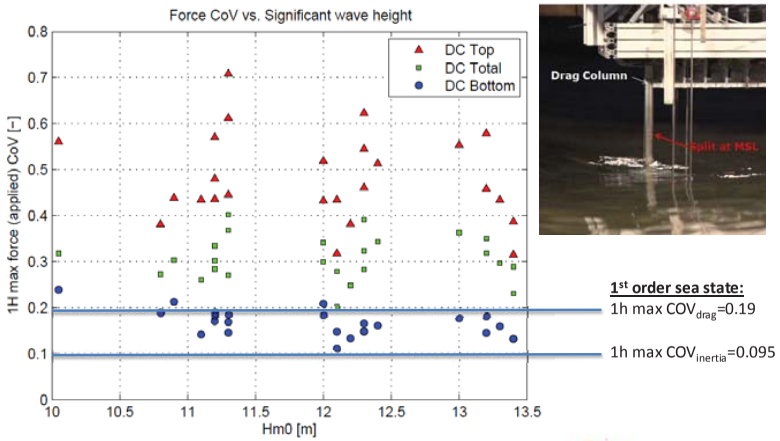
Summary of the impact on structural reliability of the findings of the Tyra Field Extreme Wave Study 2013-15

"Drag" (split at MSL) short-term COV on 1h max.



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Summary of the impact on structural reliability of the findings of the Tyra Field Extreme Wave Study 2013-15

Comparison of new response data statistics w.
Shell 90'ies SRA (Southern/Central North Sea):

Item*1	Model*2	RSR	$E20_{ratio}$*4	COV_E*5	$Beta_{20y}$*6	Pf_{annual}
Shell [2]	Na	1.73	0.84	0.212	3.25	2.8e-5
DC_low	Static	1.73*3	0.79	0.24	3.25	2.9e-5
DC_up	Static	1.73*3	0.67	0.66	1.85	1.6e-3
DC_up	Static	4.01*3	0.67	0.66	3.24	3.0e-5
DC_up	Modal 0.4hz	4.12*3	0.83	0.58	3.24	3.0e-5
DC_up	Modal 5.0hz	6.88*3	0.88	0.81	3.24	3.0e-5
DC_up	'Raw' 3.0hz	6.10*3	0.67	0.90	3.24	3.0e-5

*1: DC: Drag Column; "low/up": Lower and upper section (split at MSL)
*2: Static: Wavelet filtered; Modal: Newmark on top of 'Raw' (SDOF pulse DAF); 'Raw': As recorded low-pass filtered at 15Hz (full-scale) to remove noise
*3: RSR=Mean resistance/$E100_{Stokes5th}$
*4: $E20_{ratio} = E20_{mean} / E100_{Stokes5th}$. $E20_{mean}$ obtained from the Log-Normal fit
*5: COV on 20y max environmental load. In addition all calc's include a COV of 8% on load model (marine growth etc.) and 5% on resistance [2]
*6: Safety index (FORM) related to 20y maxima

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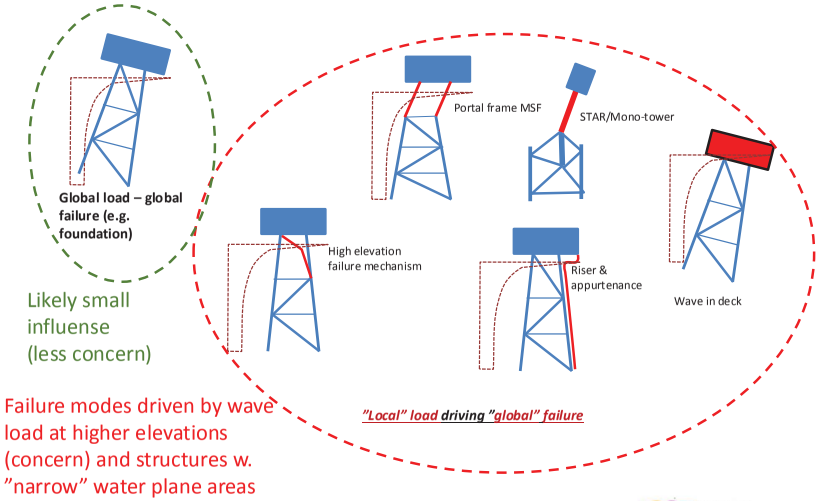

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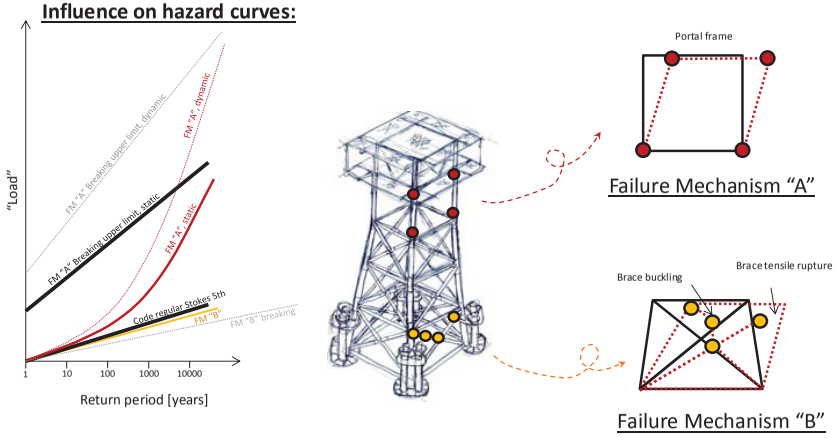
Influence on hazard curves:



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Summary of the impact on structural reliability of the findings of the Tyra Field Extreme Wave Study 2013-15

Conclusion:

Relative to current industry practice we conclude for the Tyra study (d=45m in Danish North Sea):

- Extreme crest heights will be higher ("higher than 2nd order effects")
- Many failure modes will have more onerous extreme load statistics due to extreme irregular waves (breaking)
- Different "wave types" may drive different failure modes (multi-DOF failure mechanism models req. and correlation effects)
- Dynamic response (pulse load dynamics – "ringing") will be significantly increased for many structures
- Increased risk in many structures relative to "existing" SRA
- Our conclusion: Code based analyses currently does not provide assurance of adequate structural integrity for our safety critical elements exposed to extreme wave load

The 3rd Offshore Structural Reliability Conference
OSRC2016
14-16 September, Stavanger, Norway

Summary of the impact on structural reliability of the findings of the Tyra Field Extreme Wave Study 2013-15

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ABSTRACT: In 2012, a photo was discovered of a plunging, breaking, extreme wave, occurring in a 10m significant wave height storm at the Danish Tyra field (depth≈45m). A search for further data/evidence revealed a video showing a second even larger plunging breaker (crest height estimated at 17-17.5m) in the same storm at the same field site. These observations were highly concerning as they exceeded the existing 10,000 year return period abnormal event design criteria with respect to both crest height and crest kinematics. The observations triggered an extensive study in the period 2013-15 aimed at mapping the occurrence of highly non-linear extreme wave events and clarifying the impact of these in relation to structural integrity. The work focused on the 45m water depth in the low air gap Tyra field, but it also shed light on issues that are relevant over a broad range of water depths and failure modes not driven by wave in deck loading. The core of the underlying technical development is described in a companion OSRC 2016 paper. The present paper focusses on the findings based on the extensive Monte Carlo (MC) structural reliability assessment (SRA) model that was developed. The work allows some general conclusions to be drawn on the influence of highly non-linear irregular waves and the consequence for extreme load statistics. The overall finding is that loads from non-linear extreme waves will often not play a major role at 10^{-1} annual exceedance probability, but at 10^{-3} and 10^{-4} annual probabilities (and lower) they will control the environmental load statistics and thereby also the reliability of many structures/failure mechanisms. In addition to irregular wave load effects it was found to be important that the SRA model can handle multiple failure mechanisms.

KEY WORDS: Offshore structures; Extreme irregular wave load; Wave breaking; Structural reliability, Monte Carlo, SRA

1 INTRODUCTION

The Tyra gas field is a hub for gas export and approximately 90% of Danish gas production passes through it. The first structures were installed in the field in the early 1980s. Today 11 bridge connected platforms are operating at two field locations (Tyra "East" and "West" 3km apart). Owing to depletion of the reservoir rock the Tyra field is prone to seabed subsidence. Presently, the seabed has settled approximately 5m at both field locations. In year 2000 the first platform exceeded the predicted 10,000-year return period design air gap. This triggered the Tyra Subsidence Phase I re-assessment project. This project was followed by a second phase targeting the integrity of riser systems and interconnecting bridges. Key points in this initial (re-)assessment project were:

- Design check for Ultimate and Accidental Limit States (ALS).
- The ALS employed a regular Stokes 5th order design wave based on the predicted independent 10,000-year return period crest height (16.5-16.7m including tide and surge with crest peak particle velocity of approx. 10m/s). In addition a check was included for a "worst case" 3D "Freak Wave": a deterministic event with 14-15m/s peak crest velocity was defined for this. Design waves were derived from site measurement records dating back to the early 1980s.
- Structures were strengthened to meet these criteria: addition of diagonal bracing to portal frame module support frames; some deck reinforcements; and grouting of compression bracing. This enabled in the order of 10MN horizontal wave in deck (WID) loading to be resisted.
- A large test program was performed at scale 1:26 to develop WID models applicable to the (typically open deck) Tyra structures (grating covered H beams).
- Platform deck structures were strengthened to meet at least a 3.6m deck inundation in the design WID event.
- The strengthening was supplemented by adverse weather production shut down and depressurization of equipment and prohibited access of personnel to lower decks.

Until 2012 these re-assessment criteria were considered prudent and conservative relative to general industry practice and code requirements. However, further to the discovery in 2012 of a plunging breaker in the Danish Tyra field (Figure 1, d≈45m), followed by a video of an even larger plunging extreme wave in the same storm that exceeded the 10,000-year return period design crest height, we were concerned that our assessment criteria were not adequate in terms of both the statistics of the extreme crest heights as well as extreme wave particle kinematics. This led to the initiation of two new projects: Tyra "As-Is" targeting an as accurate as possible estimate of structural reliability considering "New Knowledge" and Tyra "Future", targeting a permanent solution in the event that the original design basis was found to be inadequate.

The present paper presents the Monte Carlo (MC) based SRA methodology developed in the "As-Is" project, the main findings of the "As-Is" project and a comparison of the

present work with SRA models reported in the literature. A companion OSRC 2016 paper [1] presents the part of the "As-Is" study related to particle kinematics and load statistics for highly non-linear irregular sea-states.

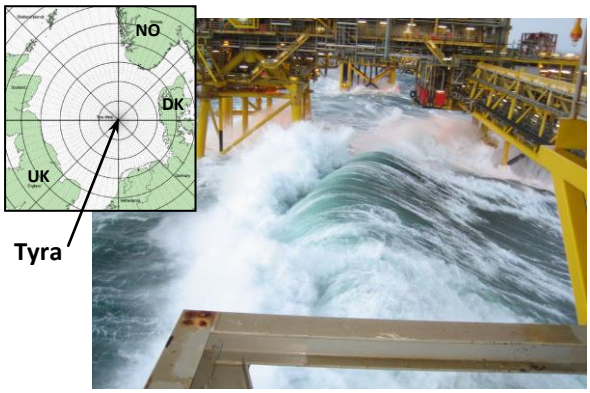

Figure 1. Tyra field, Danish North Sea, H_s=10m; T_p=14.5s. d≈45m. Insert: Field location (grid 100km).

2 FRAMING THE PROJECT OBJECTIVES

In reference 1 a detailed overview of the challenges related to consistent and accurate collapse probability estimates is given. For the best outcome the reader is advised (but not required) to read this reference before progressing. In summary, we have the following principle concerns related to commonly applied industry design approaches and SRA methods:

- Design procedures and SRA models commonly assume that regular wave kinematics (or other representations such as NewWave) uniquely define the kinematics in extreme waves. This assumption conveniently gives a deterministic link between wave height and wave load. It follows that variability of the wave load (e.g. coefficient of variation, COV, of the 1 hour maxima) in this model is solely a function of the variability of the wave height. Similarly, due to the deterministic link, it follows that the 100-year return period wave height will generate the 100-year return period load. A key concern is that extreme irregular waves, ultimately breaking waves, have large variations in particle kinematics relative to a regular wave of the same height. In the crest these irregular wave velocities may far exceed the velocities in the regular waves. Depending on the "geometry" and "elevation" of the wetted area generating the load in a given failure mechanism, in many cases the variability of wave loads is expected to be larger than commonly assumed. Hence, different irregular waves of equal height may produce dissimilar loads.
- It is common to assume that normalized statistics for global mudline loads, as represented by hazard curves for example, are representative of load statistics related to a range of failure mechanisms. It is a concern that extreme irregular waves, which will see large velocity variations high in the water column, will drive more "onerous" load statistics than seen at mudline. Actually, in a given structure, the mudline wave loads have the "narrowest" load distribution.
- Existing SRA models are either based on first member failure or ultimate system strength ("pushover" analysis). Even though each member in an offshore structure is often optimized in design, it is common to apply SRA methods assuming only a single resistance variable (a single failure mechanism). It is a concern that in an optimized structure, where multiple failure mechanisms may be loaded close to the limit of system collapse, series system effects can have a negative impact on the structural reliability. This is equivalent to the reliability of a single chain link relative to that of the entire multi-link chain. Moreover, considering irregular extreme waves it is an even greater concern that wave loading that drives different failure mechanisms will not (as in the case of regular waves) be close to fully correlated. For irregular wave loading it becomes important that all relevant failure mechanisms are assessed and that correlation effects between failure mechanisms are accurately reflected in the SRA model.

In summary, much of the concern relates to a possible lack of consistency between Structures, Met-ocean, Hydrodynamics and Reliability disciplines. For this reason overall model consistency is a key parameter in the development of the new SRA simulation model.

3 THE MONTE CARLO (MC) SIMULATION MODEL AND INTERMEDIATE RESULTS

Early in the project it was clear that none of the existing SRA approaches applied to offshore structures were suitable for our purpose. The main shortcoming relates to the link from the environmental statistics to loads and the consequence of multiple failure mechanisms.

Considering that our goal is to deliver an accurate estimate of the annual collapse probability, owing to the non-linearities involved an improved SRA set up was required. It was decided to target a Monte Carlo (MC) based solution, which, as close as possible, can capture the nature of the problem. We simulate the storms occurring in "1 million life cycles" (approximated by 1e6 realisations of 6 calendar years in the service period) to allow satisfactory convergence to the 1e4 to 3e5 range of annual return periods of interest. To link long and short-term statistics, sea-states are assumed to have stationary stochastic variation over a 1 hour duration. Hence, the storms are divided into 1 hour blocks of sea-states comprising significant wave height, peak spectral period and spreading (H_s, T_p; s_p). Only the sea-state parameters are assumed constant in each hour; associated parameters (current, wind, etc.) may vary.

The crux of our challenge is found in the model of the 1 hour sea-state. Here we need a fully non-linear, irregular, short-term simulation model which can predict the extreme load (response) for all relevant failure modes in a given structure. We analyse all extreme storm events (extreme waves and associated loads occurring in the 1 million lifecycles) which potentially can collapse our structure. Once we complete this task, the annual failure probability in each calendar year is established by counting the years with failure divided by the number of years simulated (1 million).

3.1 Requirements to the MC model

The framing called for a model which could consider accurate estimates of load and resistance as well as implement stochastic variation of all important parameters, i.e.:

- Fully non-linear irregular crest heights and kinematics
- Load distributions in agreement with the geometry and elevation of the wetted area driving failure mechanisms
- Correlations in both loading and resistance between failure mechanisms
- Pulse load dynamic response (including influence of WID loading)
- Best possible estimate of the distribution of associated loads and correlations of these with other loads to avoid conservatism in load sum statistics
- Consistent approach for including model uncertainty

3.2 Long-term environmental statistics

The long-term statistics of sea-states in the Tyra field have been derived from approximately 32 years of measurements in Tyra and the surrounding Danish fields, supplemented by numerical hindcast data to fill in gaps. As part of the present work a new extreme value distribution model has been developed and fitted to the tail of the measured sea-states. The new model is capable of simulating entire storm time-series of hourly values of significant wave heights and associated parameters such as wave direction, period and spreading, water level, wind and current speed. It is important to note that by using a storm model we account for the correlations between each 1 hour sea-state. Thus, the worst sea-states will be assembled in relatively few years through the storm they belong to. This is important as we target annual probabilities of failure, counting years with "one or more" failures.

The long-term environmental model is split into two parts:

1. A storm model that parameterizes observed storm events into weighted average values of single parameters describing the most important properties (duration; storm intensity, H_{m0}; peak period, T_p; direction; spreading; water level etc.) and also provides a method of expanding simulated storms into individual one-hour sea-states.
2. A stochastic model that includes the long-term (extreme tail) distribution of H_{m0} and conditional distributions of the associated parameters (T_p, spreading, water level etc.). Storm direction is modelled as a covariate, i.e. the distribution parameters vary with direction.

Marginal extreme value distributions of the various parameters are modelled with the Generalized Pareto distribution.

The model incorporates statistical uncertainty and physically based bounds on the various parameters. The former is incorporated by sampling distribution parameters from the estimated variance and covariance of the distribution parameters. The latter is imposed by limiting the allowed support range to within physically reasonable limits. Specifically, for significant wave heights, the support range has been limited to 21m significant wave height. Hence, sampled distributions that violate the criterion are rescaled such that they fulfil this criterion.

Once developed the model is used to expand the site data record to a 1 million year synthetic record of environmental parameters. For data consistency, non-environmental stochastic parameters such as future seabed subsidence, structural resistance, model uncertainty, etc. are included in the expansion.

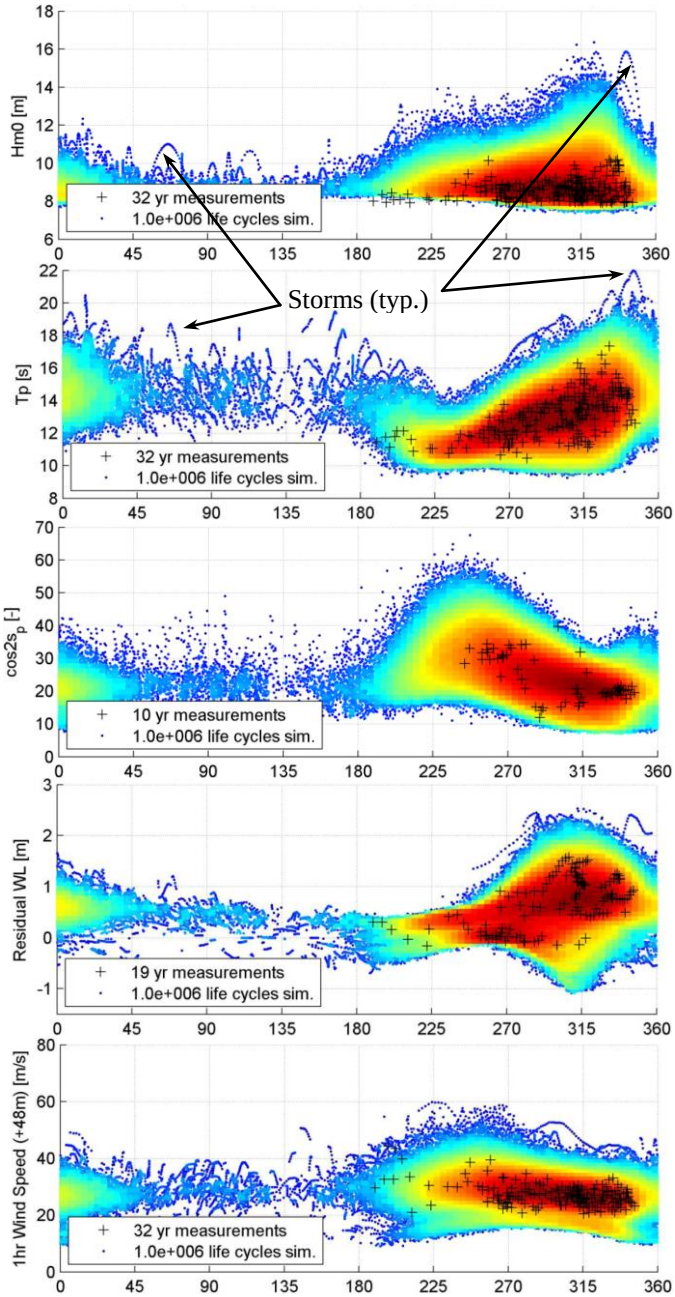

Figure 2. Plot of the 4,132,913 one-hour sea-states in the 1 million year record passing initial screening criteria *"Hs>(airgap+0.5m-tide-surge)/1.7"* for year 2016. X-axis: MWD [deg.]. Coloured dots: 1h/dot, colour = dot intensity.

To cover time dependent effects, such as seabed subsidence, and to allow the assessment to include some life cycle modelling, six independent calendar years were simulated (1999, 2008, 2014, 2016, 2019 and 2025): that is the model is

expanded into 6x1 million years. Some of the main expanded environmental parameters are plotted in Figure 2.

Without going into details, it is noticeable how the geometry of the North Sea, Figure 1, can be observed in the expanded data. The dominant wind direction is SW-W, but due to the fetch limitation imposed by the British Island (approx. 400km, ref. Figure 1) this is not the dominant wave direction; however, directional wave spreading is relatively large. The dominant extreme wave direction is NW-NNW. This is undoubtedly due to the long fetch towards the North Atlantic between Scotland and Norway for this direction. It is noted for these directions that directional spreading reduces significantly and storm surge increases as water is pushed into the North Sea.

3.3 Short-term models

The most important and also the most complex short-term stochastic model is the irregular sea-state model. The model is semi-empirical as no completely closed form fully non-linear model exists, as discussed in reference 1. Lack of applicable offshore records of non-linear irregular extreme waves [1], forces us to use experimental wave basin data as a basis for the development. Therefore a large experimental program has been undertaken, systematically testing different H_s, T_p and s_p combinations throughout the upper tail of the Tyra environmental contour (d=45m). Please refer to [1] for details.

Three test series in the test program formed the direct input for the semi-empirical model development:

- *Series "A":* Undisturbed surface and breaking. A grid of 9x9 wave gauges spanning a full-scale target area of 72x72m and video of the same area gives the reference for the 3D surface in the absence of a structure.
- *Series "C":* Re-testing Series "A" with a number of simple "response" structures present in the target area. If we accurately know the load models (in lab scale) for these simple response models, the stochastic variation of loads on the models will be a function of the stochastic variation of the underlying irregular kinematics. Hence, the simple response models inherently provide information of the stochastic variation of extreme wave kinematics driving the loads on the response models. By carefully selecting the simplest possible response model geometry which represents the problem being assessed (geometry and wetted area extent), this approach is ideal for our validation/development purpose. Further details will follow below. Three simple response models were tested, Figure 3, as part of the "As-Is" project work.
- *Series "D":* Load model development (in lab-scale). This covered a detailed wave flume program for development of lab-scale load models for the simple response structures deployed in the Series "C" test program:
  a) Development of four typical wave types in the flume. A regular wave (II), a mildly top spilling (III), a violently spilling (IV) and a plunging breaker (V).
  b) Perform Particle Image Velocimetry (PIV) to directly measure the wave kinematics in the four reference waves (II-V). Different positions along the flume were measured in the irregular events. Support the PIV with CFD data directly applying the wave maker motion as a boundary condition.
  c) Test the response models in the now known kinematics. Use the data to develop a lab-scale load model for each response test structure.

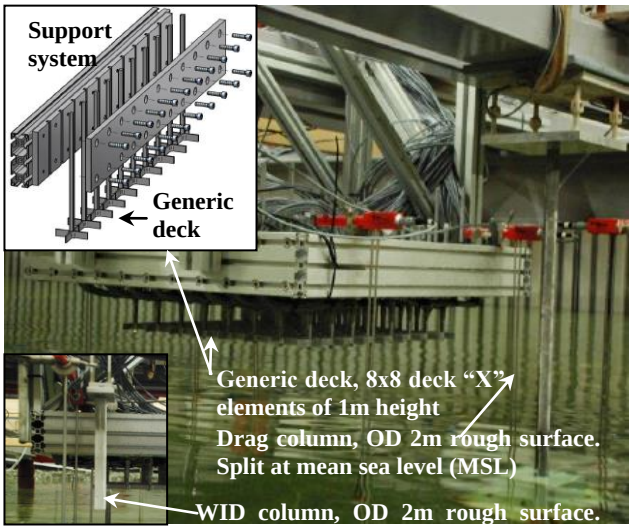

Figure 3. Simple "response models" tested.

In the companion paper [1] a more thorough outline is given of the concept of testing simple response models in many sea-states instead of testing very complex structural models in a few sea-states. Importantly, these models are simply "devices" we use to measure statistics for variation of lab-scale kinematics in a specific volume of the water column. These load records are NOT to be scaled to full-scale. Once we have used the lab-scale loads as a means of validating the lab-scale kinematics, we scale the kinematics to full scale, not the loads. In this way we make the approach independent upon possible Reynolds number inconsistencies in scaling loads.

As the reliability of the Tyra structures is controlled by the low air gap situation, the generic deck response model is given the main focus. The typical Tyra deck is a grating covered approx. 5x5m grid of 1m-high H beams. The generic deck is built from sharp corner "crosses" of 5x5x1m full-scale (see insert in upper left corner in Figure 3). Each deck cross is instrumented to deliver an individual load measurement on the cross. The generic deck was created by an 8x8 array of crosses giving a 40x40m full-size deck. By grouping different cross elements, normalized response statistics dependent upon deck size can be obtained. The deck element fixation was very stiff giving a natural frequency of approx. 40Hz (full-scale), which gave superior performance (little dynamic response) even in the most extreme plunging wave scenarios. The generic deck was developed to be the simplest possible response model that would represent the WID load process in the open deck Tyra structures. Experience from past WID model testing performed since year 2000 shows that for the deck inundations of interest, the loads generated on the deck structure were typically in the order of 70-80% of the total WID load. Hence, by developing our semi-empirical sea-state model to match the generic deck response data we have a

model which can be used to simulate the Tyra WID statistics. The deck was tested with an air gap between 12 and 14m with a complete set of sea-state tests performed for 13m air gap.

Even though our main attention was the generic deck response test model, we did manage in the tight time schedule to include three simpler response models representing load processes in different wetted area configurations in the water column. The Drag column was a vertical column split into two sections at mean sea level (MSL) allowing the normalized response statistics to be obtained on the vertical column from MSL and above as well as from MSL and down. As discussed later these data turned out to be important for evaluating the performance of existing SRA models. The last response model was the WID column. This was identical to the Drag column just mounted with an air gap of 11m representing a small volume response model/failure mechanism dominated by local crest kinematics (e.g. a riser spool).

Based on the flume test program (incl. PIV), lab-scale load models were successfully developed for all tested response models. The load models performed well across the entire range of breaking and non-breaking events (Wave II-V) making the models suitable for the purpose of isolating statistics for the kinematics.

A number of investigations were performed to evaluate the best possible set-up of the semi-empirical sea-state model. Initially a second order model with empirical corrections to the largest events to match the tests (higher order effects) was employed. However, after some work, it turned out that crest height and WID response statistics could be matched equally well applying empirical corrections to a simple 1st order FFT model (surface and kinematics). Between 40 and 50 empirical correction models were iterated against the test data before the sea-state model was deemed to be satisfactory for WID dominated failure modes.

The iterative development approach was reasonably straightforward. The sea-state matching the recorded basin conditions was set up in the numerical model (in lab-scale). The numerical simulation was now run (with random wave generation) for 10 times longer than the sea-state test. This was done to improve convergence of the simulation. The lab-scale load model was applied to the largest events in the simulation and the simulated response load statistics derived, see Figure 4. The measured (M) response data could then be compared with the predicted (P). For each sea-state, statistics of the "M/P" ratio were derived in the upper tail, see Figure 5.

This comparison was performed for all tested Series "C" sea-states for each iteration. Having a complete set of M/P ratios for all sea-states it was possible to derive the model uncertainty covering WID load (response) estimates for the developed model. It is noted that to be consistent with the analysis work performed since year 2000, the lab-scale WID model for the generic deck was developed applying the same procedures as applied to develop the load model for the Tyra structures (in year 2000). This approach was selected to limit the effect of additional WID load model uncertainties, i.e. to ensure the response data from the generic deck were as representative as possible.

With the new approach directly simulating all extreme sea-states in a long period (1 million years), the industry standard short-term distributions for wave- and crest height, for example, are no longer required as model input as they are built into the short-term sea state model which directly simulates the wave environment. Hence, these distributions effectively become internal model parameters. The accuracy of the model does not directly depend upon the crest or wave height model, but solely on how well the response of interest is predicted. Nevertheless, the empirical corrections were formulated around a new closed form crest height distribution ("Ottesen-Hansen" with crest height being a function of H_s, T_p and s_p) developed against the Series "A" test results. As the distribution is developed based on the test data, it is only applicable to 45m water depth. Model uncertainty was included in the distribution. As assurance we can calculate point crest heights at different return periods by convoluting the 1 million "year" (lifecycles) long-term sea state distribution with different short term crest distributions, see Table 1.

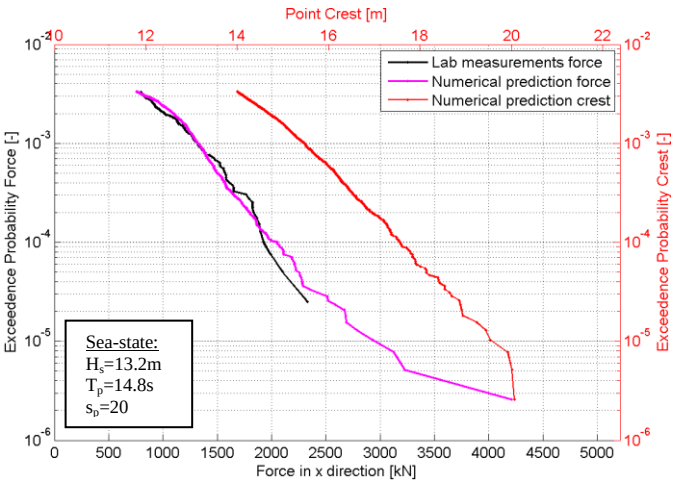

Figure 4. Example comparison of predicted and measured response data, generic deck in a 2x2 element configuration

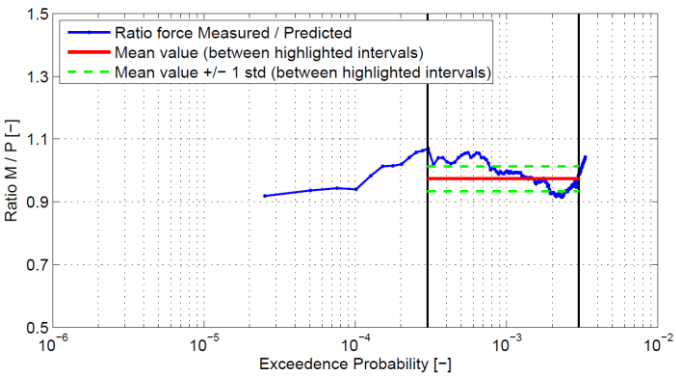

Figure 5. Measured/Predicted (M/P) ratio for the data plotted in Figure 4. The vertical lines indicate the interval applied for calculation prediction (model-) uncertainty.

The original (traditional) 10,000-year return period design crest height was 16.5-16.7m (ref. MSL), while the new long-term environmental model increases the return period point crest estimate by approximately 3m.

Table 1. Traditional environmental design statistics. SWL: Still Water Level, MSL: Mean Sea Level

Return period [years]	H_s [m]	Wave height, Forristal [m]	Wave height Boccotti [m]	Crest height Forristal [m]		Crest Height Swan [m]		Crest height Ottesen-Hansen [m]	
				SWL	MSL	SWL	MSL	SWL	MSL
10	9.6	17.7	18.3	11.2	11.8	12.1	12.7	11.8	12.4
100	10.9	20.6	21.4	13.3	14.0	14.4	15.2	14.1	14.9
1000	12.2	23.4	24.3	15.3	16.3	16.6	17.6	16.3	17.3
10000	13.7	26.3	27.3	17.5	18.7	18.8	20.0	18.5	19.7

Two thirds of this increase in the 10,000y return period crest height (2m) is caused by removal of a truncation of the upper tail of the H_s distribution, which previously was applied based on limited water depth. However, the analysis of wave records from the Southern North Sea, performed as part of the present work does not support this assumption and therefore the truncation was removed. Of more general interest is the 1m increase at 10,000y return period between a Forristall (17.5 m) and the new "Ottesen-Hansen" crest height (18.5m). As the Forristall distribution is to 2nd order, this 1m increase is a measure of the importance of "higher than 2nd order" effects.

Table 1 also lists a "Swan" crest height. This crest height distribution was made as part of project assurance independently by prof. C. Swan at Imperial College. Important to note is that prof. Swan based his work on fully independent data (and analyses) obtained in the IC wave facility, i.e. independent upon the main test program at DHI underlying the "Ottesen-Hansen" distribution. Excellent agreement on the 10,000y return period crest height (within 0.3m) is found between the two independent estimates.

3.4 Structural response and capacity

In the MC setup, for each of the 6 calendar years analysed in a life cycle, we are to evaluate all events which reach the Tyra deck structures within 1 million life cycles. With the present air gap (2016) of the lowest structure at approx. 14m (ref. MSL), we are to evaluate a large amount of events. It was soon realized that a cloud based analysis set-up was required and that this would set strict requirements to robustness in the numerical calculation tools for the approach to work.

Traditionally offshore structural analysis is performed in a deterministic format. This allows for detailed and advanced structural resistance calculations to be performed as the number of load cases is limited, i.e. typically only the 100 and 10,000-year return period waves from a few directions. Advanced non-linear analysis tools ("pushover" type) can be applied and the engineers can "nurse" these to ensure the results are correct with regards to convergence, correct failure modes, rupture/stop criteria, etc. These fully non-linear pushover analysis tools are generally not sufficiently stable to allow for bulk application in the cloud. This is a challenge, as our entire knowledge generated during 15 years of detailed assessments and reinforcement design of Tyra structures lies in these detailed structural analysis models. For success we need to develop a way to link detailed pushover resistance results to the MC without losing accuracy in the resistance estimate.

In the 1990s Shell faced the same challenge developing their SRA model [2, 4, 5]. After analysis of both detailed as well as simple load models, they realized that after normalizing the global load statistics, the response distributions were close to identical between the models. They utilized this concept to develop general conclusions about the reliability of offshore structures by analysing the simplest possible response model: the global mudline response on a 1m diameter vertical column. The basis for the Shell work will be discussed later, but here it is just important to note that simple response models can be a very effective tool to obtain detailed knowledge about normalized response statistics on very complex structures, the only condition being that the simple model "replicates" the load process (response) of interest. That is the loading on the simple model must be generated in the same volume of the water column as the detailed model and follow the same principle load process as the model it represents.

To suit our present purpose we need to re-work and further develop the Shell response model format such that it can:

- Deliver normalized statistics relevant for failure mechanisms developing at all locations in a structure.
- Identify the response level where a given failure mechanism reaches its (stochastic) capacity.
- Execute quickly and reliably.

We succeeded to develop this new "Bar & Membrane" (B&M) model and apply it to all SRA analyses of all Tyra structures. It comprises two parts:

- A fast running and reliable load model based on a calibrated assembly of Morison point load elements.
- A simple response model mapping the point loads to the main failure mechanisms.

The B&M model is basically an interface model which bridges between the very high number of extreme events assessed in the MC analysis and the very detailed, but cumbersome, deterministic structural analysis of much fewer, but representative extreme events. It works by relative comparisons of subsets of actions between different analyses.

3.4.1 The equivalent point load model

A simple, quick and robust equivalent load model is prepared for each structure. The point elements are arranged to cover the spatial extent of the structure. In most cases jackets can be modelled by rows of vertical elements typically located in the centre of the dominating structural elements (legs, conductors, etc.). Less dominating elements (braces, appurtenances etc.) are lumped to the closest line overall balancing the loads. Point elements arranged along vertical lines are preferable, as this will ensure the most numerically efficient calculation of kinematics in the FFT that underlies the semi-empirical wave model, see Figure 6. The equivalent model is checked against the full model for a number of detailed load vectors.

The equivalent load model is similar to the Shell "stick" models detailed in [4, 5]. A load mapping procedure is built into the model. Each Morison point element maps the load to a centre node location. Above a given horizontal plane, the

sectional loads are the sum of the loads in the mapping nodes. A similar approach is used in the topside for WID loading (and wind, gravity etc.). For WID the spatial load distribution (and 3D crest shape) effects are more important and for this reason the point elements are spread throughout the topside in a high number, i.e. not assembled in a few vertical lines.

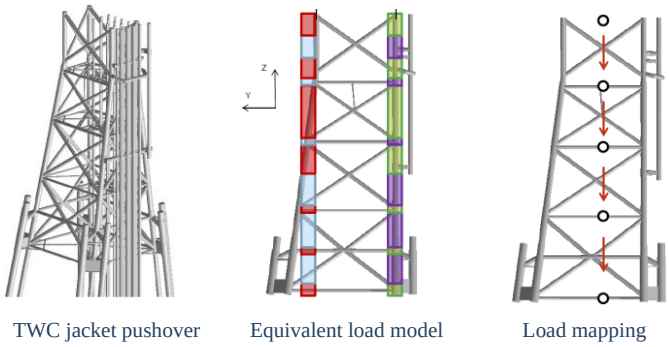

Figure 6. Equivalent load model, only jacket load shown.

3.4.2 B&M response model

The B&M format evolves along with the load mapping. A jacket structure is globally a cantilevered beam. On an overall system level the section (elevation) loads are statically determinate. Possible non-linear internal force flow in a jacket is caused by statically indeterminate (over-constrained) bracing systems "internally" in the jacket. For example, an X-braced jacket has two load paths through each X-bracing which normally allows for a large force redistribution in the bracing as the X-brace failure mechanism approaches its ultimate capacity. At the point of collapse, when the failure mechanism is fully developed, the structure can no longer redistribute loads: alternative load paths are exhausted and/or a structural component suddenly loses capacity due to fracture, and collapse occurs. Here it is important to note that the response of individual structural members may be highly non-linear in statically indeterminate bracing systems, but at failure the loads on each mechanism will distribute as in a statically determinate structure. This allows us to use a simple force equilibrium concept to map the global loads uniquely to each main failure mechanism in the structure.

Taking a typical 4-legged jacket with external piles as an example, see Figure 7, failure mechanisms can develop in each "bracing bay" either in one of the legs (4 off) or in one of the 4 diagonal systems. The legs will primarily resist the vertical loads (gravity and overturning moment) and the diagonal bracing will support horizontal shear and torsion (around a vertical axis). Hence, the failure mechanism of each bay can be considered as an assembly of 4 bar elements (the legs) and 4 membrane elements (X-diagonals). In each bay, loads distribute to the 8 elements following simple force equilibrium principles. This is the Bar and Membrane (B&M) model principle. Each bar and membrane assembly "element" represents a potential failure mechanism and loads are shed between the elements following simple equilibrium considerations. This is arranged very robustly in a set of simple influence functions mapping the (sectional) loads from the mapping nodes to the B&M elements. The principle is sketched in Figure 7.

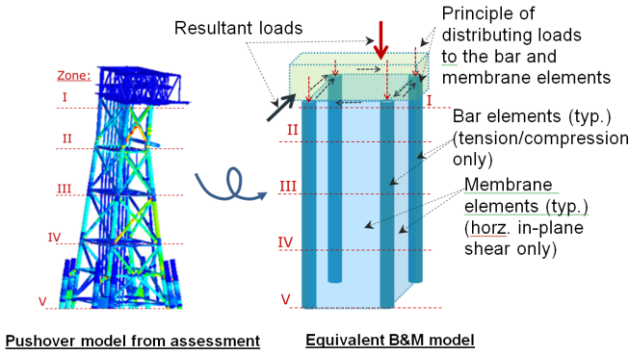

Figure 7. Sketch of the B&M format.

It follows that for any wave, current, wind and gravity load vector we can calculate the load and map this to each (B&M) failure mechanism.

The final step in the development is to assign a stochastic "capacity" to each B&M failure mechanism. This is the all-important link giving us access to the 15 years of detailed structural analyses of the Tyra structures. For each structure we have a detailed pushover model and a calibrated Morison point load equivalent B&M model. We now run a matrix of pushover analyses on the detailed model pushing the load vector towards expected worst case load events. This covers regular long period waves, plunging breaking waves, worst case gravity cases, extreme wind etc. all performed with the aim to trigger as many failure mechanisms as possible in the structure without being unrealistic. From the pushover analyses we note the position of the failure mechanism and the load vector at collapse. In the next step we transfer the loading conditions at collapse to the B&M model, i.e. we apply the same wave kinematics, current, gravity, wind combination to the B&M model, and map the load to each failure mechanism. We mark the failure mechanism which matches the failure mechanism triggered in the pushover analysis. This procedure is repeated throughout the principal load cases in the analysis matrix.

For the weakest failure mechanisms in the structure we will have many B&M load cases matching collapse in the pushover, for others we will have a few and for some (the strongest) mechanisms we have not been able to trigger failure. For each B&M mechanism we calculate the COV and mean of loads matching a pushover failure mechanism. In a typical jacket this COV is surprisingly low, typically less than 5% low in the water column and less than 10% at sea level, giving a strong indication that the B&M format is working as intended. We include the stochastic variation as model uncertainty on the B&M resistance model and duly correct for the limited number of observations. For failure mechanisms that are only triggered once or not triggered at all we take the highest resistance value as a deterministic, lower bound capacity. If failure occurs in any of these in the SRA, we should return to the basic load cases applied to the pushover model and update/extend the analysis matrix. We now have a calibrated B&M model that is robust, fast and reliable.

A final comment relates to dynamic response. The B&M model only maps the static loads. A stochastic pulse load DAF model is built into the load model. This model is an extension of the deterministic model applied for the previous WID assessments of the Tyra structures. The model accounts for the actual shape and duration of the WID load, the natural period of the jacket and the non-linear deformation capacity.

3.5 The simulation setup

The main components in the MC model are now available:

- A primarily environmental record of 1 million life cycles (6 years throughout the service period) comprising 1 hour sea-states and associated environmental parameters such as wind, tide, current surge etc. Conveniently, other stochastic parameters are also included, like model uncertainties, subsidence forecast, sea level rise (global warming), failure mechanism stochastic resistance variation bias, WID DAF, etc. All stochastic variables are categorized to either have "event", "hour", "annual" or "life-cycle" stochastic variation indicating how often a new realization of the variable needs to be drawn in the MC simulation.
- A semi-empirical sea-state model calibrated to deliver good WID load statistics and duly accounting for model uncertainties.
- A fast running and stable response model which can assess individual failure mechanisms and correlations in loading between failure mechanisms (system collapse).

The MC simulations are arranged as outlined in reference 1. First, identify all sea-states capable of producing a WID event and for these make a realization of the 1 hour sea surface below each deck structure (deck area); identify extreme waves reaching at least 0.5m into the deck. For each of these events calculate the stochastic input to the response model, map loads to each failure mechanism and compare load with resistance and mark if failure occurs.

The simulation is built around a simple ASCII file format. For each 1 hour sea-state a single row of input (sea-state random seed, stochastic realizations, load and resistance), i.e. every parameter in the model is assembled. This arrangement is optimal for parallelization in the cloud, as each 1 hour sea-state in principle can be launched on different CPUs. The extent of the analysis work was however massive. At the time of execution it was the largest cloud computing job running in Europe with up to 30,000 kernels assigned.

The new format turned out to be very powerful enabling inclusion of almost any stochastic variation and non-linearity (as long as we could describe it) and not least for assessment of correlation effects. To maximize the value of the simulation, each Tyra structure was placed at the correct grid location and a sea-state realization was progressed over the entire field grid so if a given wave event would hit several structures, we model this correctly in the simulation. This set-up allowed us to determine the failure probability of individual mechanisms, the system failure probability (collapse of one or more failure mechanisms in a structure) and also the "field" failure probability, wherein the probability of losing one or more structures in the field, can be assessed.

4 SRA RESULTS

The SRA results for the 4-legged Tyra East B (TEB) wellhead platform (similar to jacket sketched in Figure 7) are used as an example. This platform has previously been extensively strengthened to meet the 3.6m inundation in the year 2000 design basis. This involved adding new bracing to the original portal frame below the deck, significant strengthening of the deck structures and grouting of legs and selected jacket braces to increase compression capacity. The strengthening enables the jacket to resist a WID event in the order of 1000t horizontal load. The B&M model is based on analysis of this strengthened structure. This must be kept in mind when considering the results that follow: if no strengthening had been performed, the results would be significantly different.

In Figure 8 we plot the collapse probability using level shear and level overturning moment as our reference. Thus we define collapse if shear or overturning moment OTM in the main mapping nodes exceeds the capacities calibrated against the TEB pushover model. Each of the lower 5 curves in Figure 8 provides the failure probability at a given level of the platform with the lowest blue curve representing failure at the mudline.

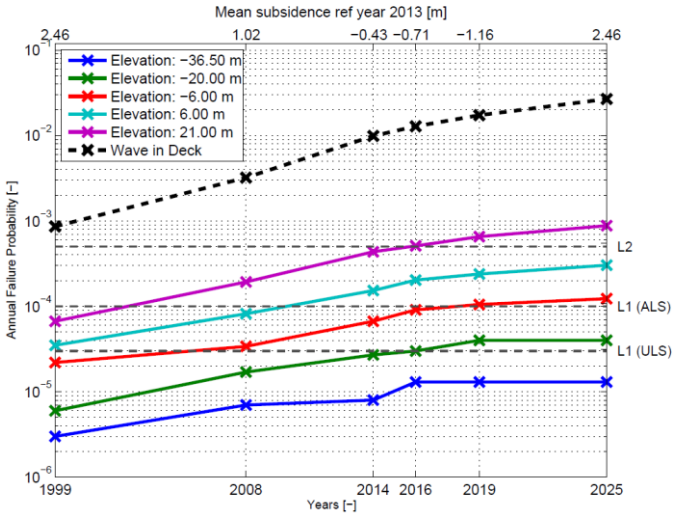

Figure 8. TEB SRA based on level loads (BS and OTM). Note original elevation ref's, i.e. 21m is bottom of deck.

The plot in Figure 8 and the following plots all have the annual collapse probability along the log-scaled vertical y-axis and the service year along the linear lower x-axis. The upper x-axis shows discrete values of the average seabed subsidence relative to year 2013. The development in failure probability throughout the service period is primarily driven by the progressing seabed subsidence resulting in a reduced air gap. The dotted black line gives the probability of a wave reaching the deck structures, i.e. the "area" WID statistics.

A noticeable conclusion from Figure 8 is that using the mudline global loads as the single response parameter (lower blue line), the platform actually complies with the 3e-5 "L1" criteria outlined in references 2 & 3. However, as we increase the reference elevation for the shear and OTM loads, the collapse probability increases. At the highest elevation the collapse probability has increased by around two decades

relative to the mudline value. It is important for general conclusions to recall that the platform has been strengthened primarily in the upper sections to resist the previous design load: the shift in failure probability would be even more dominant if we had assessed the unstrengthened structure.

If we define collapse as failure in one or more B&M elements, i.e. the total TEB "system" collapse probability estimate, we obtain the results plotted in Figure 9. In Figure 10 all individual system results for the Tyra East structures are plotted as well as the combined field result.

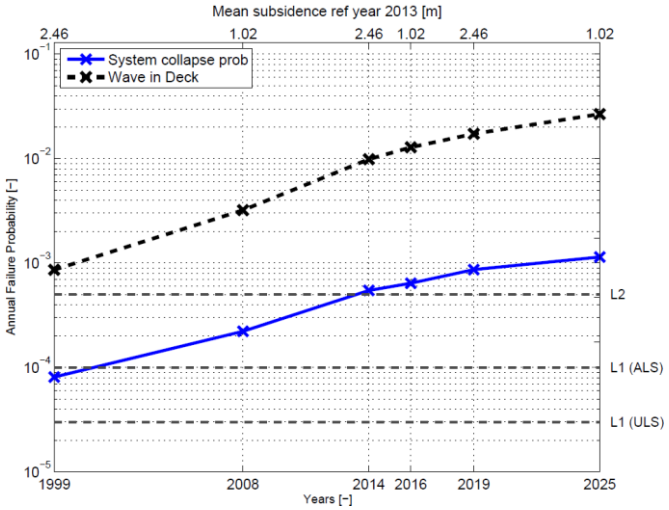

Figure 9. Annual TEB collapse probability as function of developing sea bed subsidence.

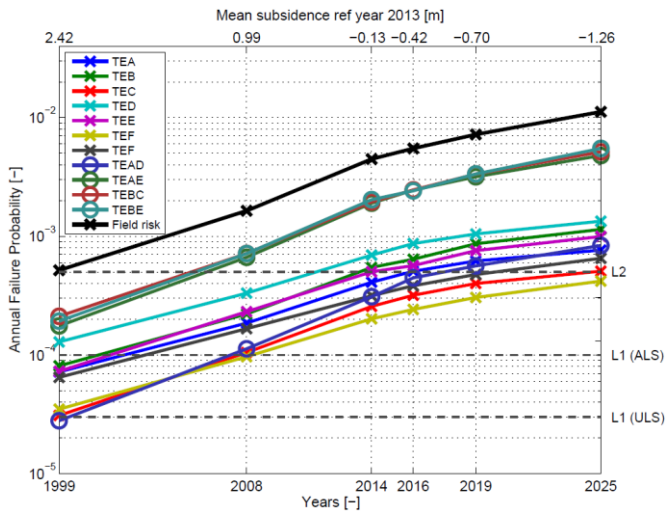

Figure 10. Individual platform annual collapse probability estimates for Tyra East. Black curve is the "field" risk.

For TEB the system reliability curve, Figure 9, is only slightly higher than the failure probability of the upper bay, Figure 8. In other words it is failure of the upper bay which controls the reliability of the platform. In Figure 9 it is seen that the system reliability curve is close to a parallel shift of the dotted upper curve giving the WID probability. If we subjectively judge the subsidence difference between the two curves using the upper x-axis, we can see that the average deck inundation (3D area max. crest height) is around 3m when collapse occurs. Looking at the vertical spacing between the two curves in Figure 9 it is seen that approximately every 20th WID event will cause a global collapse of the platform. This large resistance to WID is a result of the extensive strengthening performed.

In Figure 10, the highest individual system collapse probability is found in the 100m long inter-platform bridges. Although the air gap of these is approx. 1.5m higher than the air gap of the supporting deck structures, even for small inundations of the bridge the WID loading will control the reliability. For these structures the probability of WID and the system collapse probability curves are close to each other.

The upper black curve in Figure 10 is the "field risk" curve, the annual probability of collapse of "one or more" structures at the Tyra East field location. It is noticeable that the field risk curve is significantly higher than the underlying system curves. This is driven by a high spatial variation in the short-term loads.

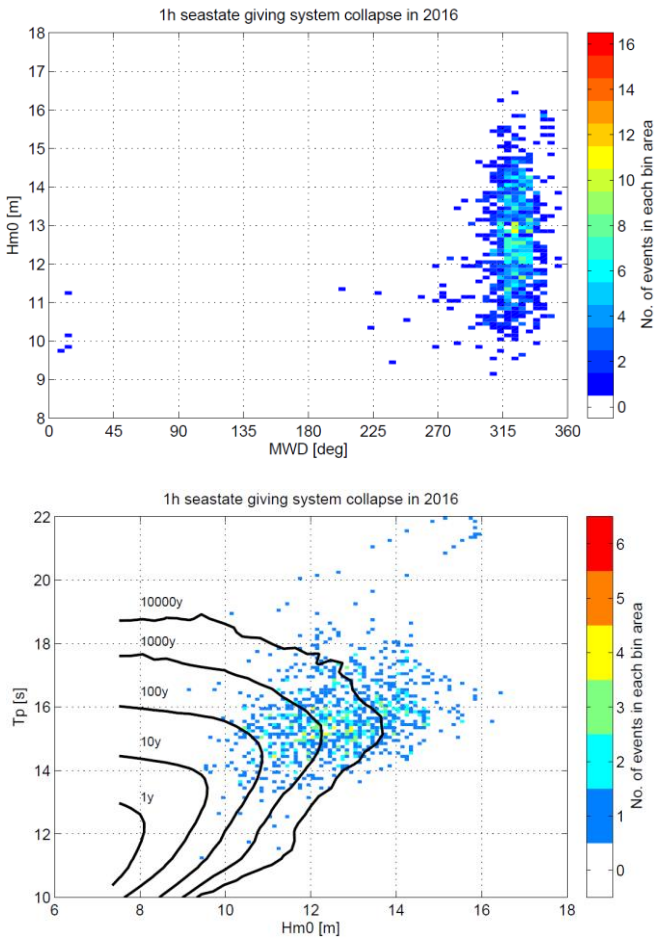

Figure 11. Plots of sea-states giving system collapse.

In Figure 11 we plot the TEB 2016 sea-states giving system collapse against mean wave direction (MWD) (upper plot) and on top of the environmental contour (lower plot). It is noticeable how the NNW direction is dominating. In relation to the environmental contour H_s seems to be the dominating factor. A bit more of a surprise is that the steeper sea-states, where breaking is more pronounced, are not contributing

significantly to the collapse probabilities. Subjectively, it seems that the sea-states with longer T_p, (i.e. those which can create a relatively large volume of water at high elevations), are a dominating factor for critical response of Tyra platforms.

5 IMPLEMENTING SEASONAL VARIATION

During the course of the project the long-term statistics were extended with seasonal variation as a second co-variant. This adds to the complexity of the long-term model, but it adds a powerful feature to the simulation. By having a "storm date" added to each storm in the database (and thereby a date tagged to each 1 hour sea-state), the seasonal variation of the collapse probability is known. These data have been used to plan mitigating activities in the field in the winter season.

6 MODEL ASSURANCE

As an independent assurance activity Imperial College (IC) (prof. C. Swan) built a detailed model of the topside of the TWC well head platform. This model was tested for 240h in a sea-state with H_s=12m; T_p=15s and s_p=20, corresponding approximately to 1,000-year return period sea-state conditions in the middle of the environmental contour (the most likely sea-state to generate a collapse for many structures, ref. Figure 11). A Q-Q plot of the WID loads from the test has been included in Figure 12.

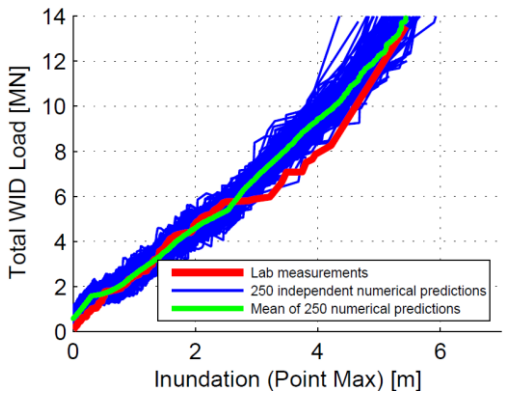

Figure 12 Q-Q plots of blind test validation.

A blind prediction of the IC test was prepared with the SRA model. The analytical WID load model was set-up following standard procedures as applied for Tyra structures and a B&M model prepared. The test was then simulated 250 times to determine stochastic scatter. The simulations are the blue curves in Figure 12. Very good agreement between test and simulation based on a WID response model is observed. For further details related to general project assurance, please refer to reference 1.

7 EVALUATION OF SHELL'S 1990'S SRA MODEL

The most commonly applied offshore structure SRA methodology today is based on work by Shell in the mid 1990s [2, 3, 4, 5]. The principles of this model were also applied in the work performed by the Bomel-headed JIP [6, 7] targeting reliability of North Sea structures. A summary of this, relevant to the present work, can be found in the companion paper [1]. Below we compare our new knowledge of response statistics against the existing SRA model as outlined in reference 2. We concentrate on conclusions and assume that the reader is familiar with the Shell model.

Shell based their model on the global load (response) on a vertical column. By normalization of the calculated load statistics (for historical reasons they used the distribution of 20 year maxima), they derived SRA results to support the selection of RSR values as well as supporting the setting of partial action factors. The present work provides an opportunity to use wave basin test results directly to evaluate the theoretical assumptions underlying the Shell model.

As outlined earlier and detailed in reference 1, the Tyra "As-Is" work scope included testing a vertical column (the "Drag column", Figure 3) split at MSL, across a matrix of non-linear irregular sea-states. This work allows us to effectively replace Shell's theoretical load calculations [2, 3] with wave basin recorded response data. By repeating the same procedure for both the upper and lower part of the Drag column independently, we have the option to compare directly with the Shell model as well as doing relative comparisons of statistics between the two sections of the column. The approach applied is briefly as follows:

- For each sea-state fit (least squares conditioned to same COV and mean as the sample) a Gumbel distribution to 1hr response maxima. This is our short-term statistics.
- The long-term response statistics are established by numerical folding with the sea-states in the "1 million year" (life-cycles) file. For each 1hr sea-state above 9.8m (H_s) we locate the closest test and make a draw of a 1hr maximum response from this test. We now have a 1 million year record of 1hr maximum response on each column section.
- Using maxima in 20 year bins, we follow Shell procedures and fit a log normal distribution to the tail of the data. We only fit to the upper tail between the 10^{-3} and 10^{-5} annual exceedance probabilities where the log-normal distribution provides a good estimation. We calculate COV (COV_E) and the mean ($E20_{mean}$) of the 20-year maxima loads.
- Using our lab-scale load model and the 100-year return period wave height from our long-term statistics, we apply the Stokes 5th order theory (no kinematics factor) to calculate the 100-year regular design response ($E100_{Stokes5th}$). The ratio ($E20_{ratio}$) between the fitted $E20_{mean}$ and the design $E100_{Stokes5th}$ is the mean value of the 20-year maximum response brought to the same scale as the resistance expressed by the RSR value. We can now use the new response statistics in the Shell model.

The results, Table 2, are interesting. For the Tyra location (Southern to Central North Sea), Shell [2] found a COV of 0.212 of the 20-year response maxima giving a required RSR value (load factor on E100) of 1.73 to reach an annual failure probability of 3e-5. The model fit to the lower column data in Table 2 is spot on the Shell model. Good agreement is expected for the lower column section as the response is not affected by any wave loads above MSL, i.e. we are close to matching the Shell assumption of short-term load variations in linear sea-states. Even though our environmental response (E) is based solely on wave load while Shell included

contributions from other environmental factors such as wind and current, with a slightly less steep hazard curve, the agreement for the lower column, between two fundamentally different approaches, is remarkable. This adds trust in the results when we later do relative comparisons between the lower and the upper column sections.

Table 2. Comparison with Shell SRA.

Item*1	Model*2	RSR	$E20_{ratio}$*4	COV_E*5	$Beta_{20y}$*6	Pf_{annual}
Shell [2]	Na	1.73	0.84	0.212	3.25	2.8e-5
DC_low	Static	1.73*3	0.79	0.24	3.25	2.9e-5
DC_up	Static	1.73*3	0.67	0.66	1.85	1.6e-3
DC_up	Static	4.01*3	0.67	0.66	3.24	3.0e-5
DC_up	Modal 0.4hz	4.12*3	0.83	0.58	3.24	3.0e-5
DC_up	Modal 5.0hz	6.88*3	0.88	0.81	3.24	3.0e-5
DC_up	'Raw' 3.0hz	6.10*3	0.67	0.90	3.24	3.0e-5

*1: DC: Drag Column; "low/up": Lower and upper section (split at MSL)
*2: Static: Wavelet filtered; Modal: Newmark on top of 'Static' (SDOF pulse DAF); 'Raw': As recorded low-pass filtered at 15Hz (full-scale) to remove noise
*3: RSR=Mean resistance/$E100_{Stokes5th}$
*4: $E20_{ratio} = E20_{mean} / E100_{Stokes5th}$. $E20_{mean}$ obtained from the Log-Normal fit
*5: COV on 20y max environmental load.
*6: Safety index (FORM) related to 20y maxima In addition all calc's include a COV of 8% on load model (marine growth etc.) and 5% on resistance [2]

The close agreement with the Shell model assumptions is confirmed looking at the COV of 1hr maxima calculated for each sea-state tested, see Figure 13. The lower column section fits the expected COV variation between 0.095 (inertia dominated) and 0.19 (drag dominated) stated in reference 2.

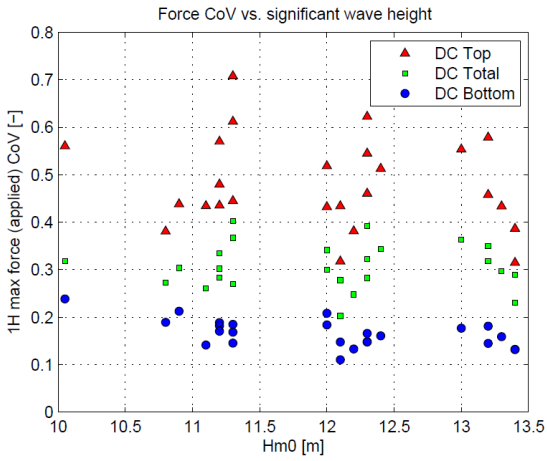

Figure 13. COV of 1hr maxima Drag column. Each dot represents a Series "C" sea-state test.

For the upper column section, Table 2, the combined long- and short-term static response COV (0.66) is 2-3 times higher than for the lower column (0.24) and the assumption in reference 2 (0.212). This increase is driven by the influence of the irregular kinematics and higher order wetted area (surface) effects. The increased short-term COV on the upper column leads to significantly changed SRA results, ref. Table 2. For RSR=1.73 the annual collapse probability increases from 3e-5 (Shell model) to 1.6e-3, a factor of 50. To meet the 3e-5 annual collapse probability for static response for the upper section we will need to "design" for an RSR value of 4.01, i.e. 2.3 times higher RSR than required for the lower column.

It was found that the 20-year maximum response value for both sections is reasonably well predicted by a Stokes 5th order wave, i.e. irregular effects do not seem to play a significant role in the core part of the 20-year maximum load distribution. In other words, extreme irregular events are too rare and too spatially distributed to significantly influence the average 20-year maximum response. However, at longer return periods, where the tail of the load distribution is more important, failure may be triggered by highly nonlinear events, especially at locations high in the water column. "Narrow" structures are more at risk than "wide" structures as a larger fraction of the loaded area can be wetted by a non-linear extreme wave, i.e. larger influence of the locally very high breaking wave kinematics. Hence, if we want to make structures safe for these failure mechanisms to say 10^{-3} or lower annual failure probability level, we need to design for larger wave loads. In terms of hazard curves, this means that the curves are much steeper than commonly assumed in the literature, see references 3 and 5 for example. In fact the curves are not straight (exponential load distribution), but curved upwards ("concave"), indicating the increasing influence of the irregular extreme waves in the upper tail.

It is stressed that the above comparison relates to the direct static load in single failure mechanisms; pending the actual structural system the response to irregular wave loads might be even greater considering dynamics and multiple failure mechanisms (system effects).

The static response was obtained by filtering the recorded raw response data. The natural full-scale frequencies of the Drag column were 3Hz (upper) and 5Hz (lower). Exposed to regular wave loading these structures would show a quasi-static response. Purely static response was observed in the response data for the lower section. However, the upper section showed significant dynamic response to extreme events in the irregular sea-state tests, indicating that extreme irregular crests do contain a large amount of high frequency energy, which can trigger pulse load dynamics far away from the peak period (T_p) of the sea-state in which the event occurs.

Several filter approaches were tested to derive the static response in the upper section. Low pass filtering was not an option as this removed significant parts of the high frequency load content. A wavelet filter was finally selected as a Newmark time integration of the wavelet filtered load pulses, applied to a linear oscillator tuned to the natural frequency of the test model, returned response very close to the 'raw' data statistics.

We can access the statistical importance of pulse load dynamics in two ways: (1) directly analyse the 'raw' response data of the column having a natural frequency of 3Hz and compare this to the quasi-static pulse load; or (2) apply the static pulse load to a linear oscillator representative of different structural types and obtain the dynamic single degree of freedom response using Newmark integration. The results are shown in the last rows of Table 2.

It is seen that over a wide range of natural frequencies (0.4 to 5.0Hz), representative for many jackets, riser systems etc., there is a significant influence of pulse load dynamics in the response to irregular extreme waves. Using the 'raw' data directly, representing a "narrow load area", dynamic, 3Hz full-scale response model, the RSR required to meet the 3e-5 criteria is approximately 50% higher (RSR≈6) than the static

value. Implementing system ductility (if present in a structure) in the otherwise elastic amplification will of course reduce the difference, but the results clearly indicate dynamic response to single extreme non-linear waves may be significant and will be an important issue to consider.

8 FINDINGS AND FUTURE WORK

As results from the Tyra "As-Is" work started to evolve during 2014, it became clear that the "New Knowledge" would likely have a negative impact on our past understanding of the integrity of the Tyra structures, resulting in a situation that was outside the acceptance criteria for manned operation during adverse weather. As a consequence, jack-up rigs were spudded at the Tyra East and West fields during the 2014/15 winter season, to ensure that the installations could be properly shut down and de-manned safely during adverse weather.

However, the current situation is not sustainable. Consequently, Maersk Oil issued a notification to the gas market early April 2016 announcing plans to cease production from Tyra East and Tyra West by October 2018. Together with the DUC partners, Maersk Oil is evaluating long-term viable solutions for safe recovery of the remaining resources at Tyra.

Based on current knowledge we see the general findings to reach far outside the low air gap situation in the Tyra field. In general we see that existing industry methods cannot correctly estimate the failure probability for collapse mechanisms developing at high elevations in the water column. A new project (Ab-normal Wave Assessment and Risk Evaluation - AWARE) has been initiated to expand the Tyra study to cover a water depth range of 30-65m and include the assessment of integrated non-linear (jacket/riser/conductor) models of the remaining 41 non-Tyra structures present in the DUC Danish concession area. The work will include a large wave basin test program covering a larger amount of principle response test models and will also extend the numerical response model (B&M) to include dynamic response of dominant vibration modes. This work will not only target a full SRA model of all structures, but also development of load combinations together with calibrated safety factors that can be applied for standard deterministic engineering assessments.

9 CONCLUSIONS

We reached the target of our study and estimated the probability of collapse for structures in the Tyra field considering the loading from fully non-linear irregular waves. The results point to non-conservative shortcomings in existing industry practice assessment and SRA methods. Relative to existing practice we see:

- Many structural failure mechanisms will have more onerous extreme load statistics due to wave breaking and failure mechanisms dominated by wave loads generated at high elevations in the water column are expected to be influenced.
- There is not a deterministic link between wave height and wave load as inherently assumed when applying regular wave theory. The large transient and spatial variability in non-linear irregular extreme waves gives a high stochastic variability of kinematics for waves of equal height. A result of this variability is that loads in different failure mechanisms are not fully correlated. This makes it very important that SRA methodologies used to estimate jacket collapse probability evaluate all failure mechanisms which can lead to global system collapse and duly consider the load correlation effects.
- Higher than 2nd order irregular wave effects may influence crest height statistics, i.e. effects beyond the generally applied Forristall crest height distribution.
- In terms of hazard curves many structures will see much steeper curves than assumed in current industry practice.
- Irregular extreme waves can induce significant pulse load dynamic response. This may affect structures (jackets, risers, etc.) which normally will have a quasi-static response to regular waves.
- Even though the study is performed for a 45m North Sea location we do expect that many of the results/conclusions reached extend far outside the range of the study. We would expect that many conclusions would apply to all harsh weather environments.

ACKNOWLEDGEMENTS

Over the duration of the project a team of external advisors provided significant guidance and supervision to the project team. To be highlighted is the assistance provided by prof. C. Swan from Imperial College. Without his in-depth knowledge and experience related to extreme irregular waves and associated loads it would not have been possible to perform the work within the time frame. Also a great thanks to prof. J. D. Sørensen and prof. M. H. Faber for ensuring consistency in the simulation methodology and to dr. Graham Stewart for performing an overall project assurance review. Finally, special thanks to the DUC partners (Shell, Chevron and Nordsøfonden) for active and valuable contributions in technical workshops, etc. and not least for supporting the publication of the present work.

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[3] M. Efthymiou, J.W. van de Graaf, *Reliability and (Re)assessment of Fixed Steel Structures*, 2011 OMAE conf. paper, OMAE2011-50253

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5.2 Presentation & article by Chris Swan, Mohamed Latheef & Li Ma

NTNU Norwegian University of Science and Technology

International Association of Oil & Gas Producers

The loading & reliability of fixed steel structures in extreme seas: recent advances and required improvements

Chris Swan, Li Ma and Mohamed Latheef
Fluid Mechanics Section
Department of Civil & Environmental Engineering
Imperial College London

The 3rd Offshore Structural Reliability Conference
OSRC2016
14-16 September, Stavanger, Norway

Overview: key questions

When estimating applied loads and failure probabilities

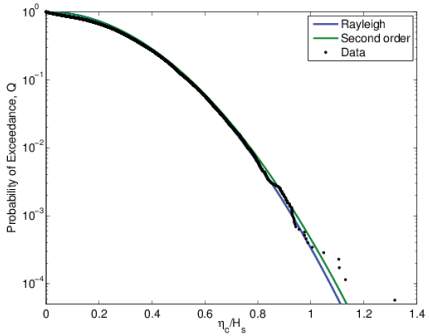
1. Do we need to consider effects above second-order (crest heights)?
2. Are area effects important?
3. What is the role of directionality?
4. How significant is wave breaking?
5. Are existing kinematics models adequate?
   - sub-structure loads
   - wave-in-deck loads
6. Given new physical insights:

   Is there is need for new / revised procedures appropriate for design / re-assessment?

7. How do these effects vary across the broad range of jacket structures (with varying effective air-gaps)?

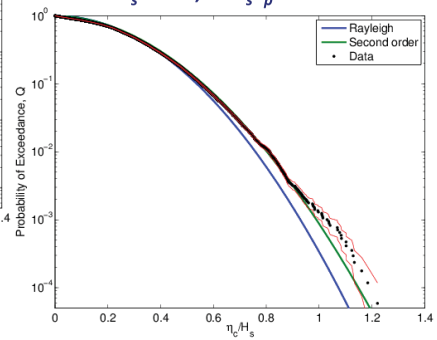
Crest height statistics

Laboratory data (T_p=16s, σ_θ=15°, k_pd=2.0)



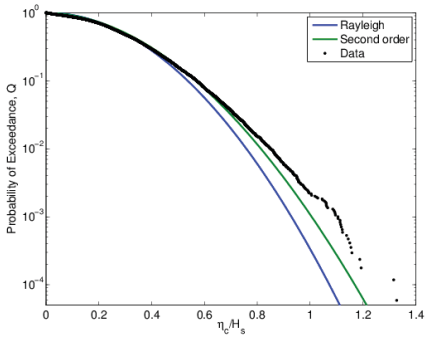
H_s=3m, ½H_sk_p=0.024

H_s=10m, ½H_sk_p=0.081



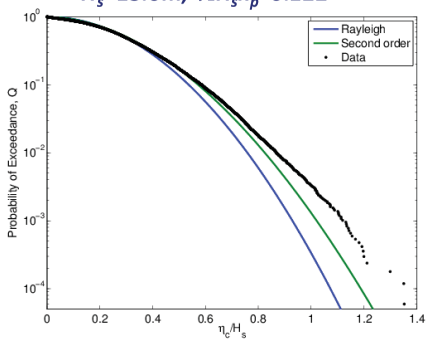
Crest height statistics

Laboratory data (T_p=16s, σ_θ=15°, k_pd=2.0)



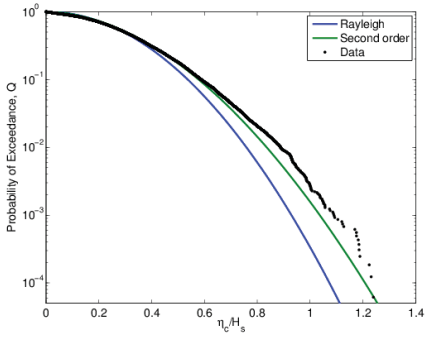
H_s=12.5m, ½H_sk_p=0.102

H_s=15.0m, ½H_sk_p=0.122



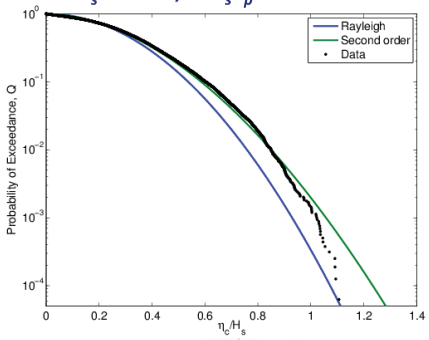
Crest height statistics

Laboratory data (T_p=16s, σ_θ=15°, $k_p d$=2.0)



H_s=17.5m, ½$H_s k_p$=0.142

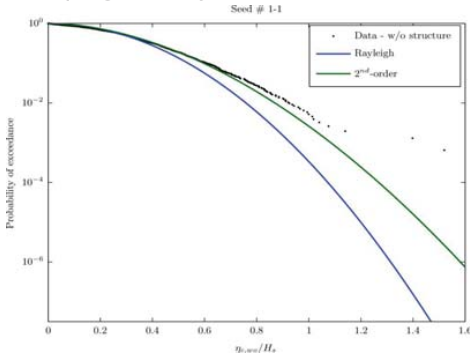
H_s=20.0m, ½$H_s k_p$=0.163



Additional crest height observations

Very long random simulations:

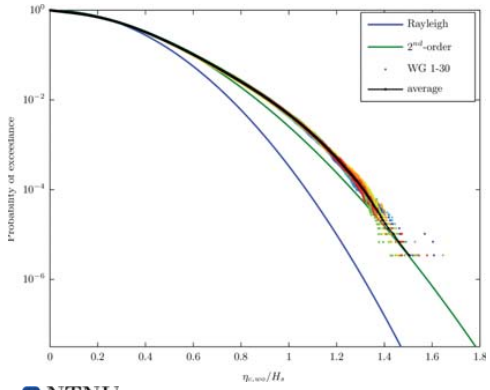
- 192 (4.8 hour) seeds ≈ 927 hours (16 times longer)
- T_p=15s, H_s=12m, σ_p=18°, Ewans spreading
- Evolution with progressively more seeds

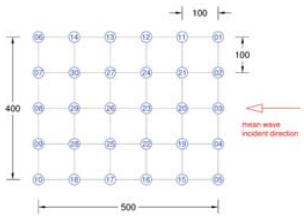


Are these effects real & representative?

(i) Spatial Homogeneity

- Data from 192 x 4.8-hour seeds
- Variability across 30 gauges

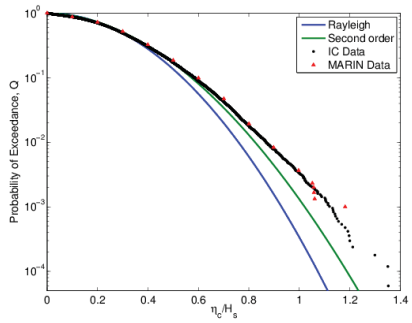


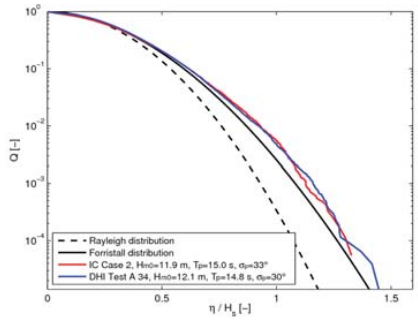


(ii) Comparisons to other laboratory data

(a) In deep water with Marin (from the *SHORTCREST* JIP)

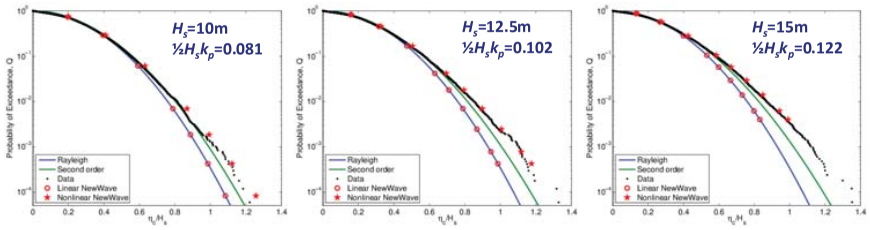
(b) In an intermediate water depth with DHI (from Mærsk's Tyra project)



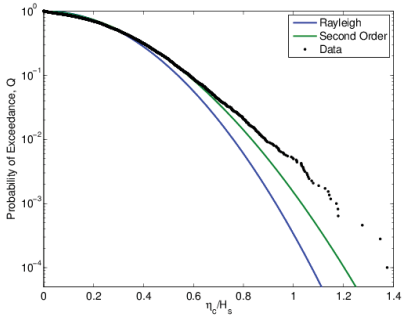


(iii) Comparisons to numerical (BEM) calculations

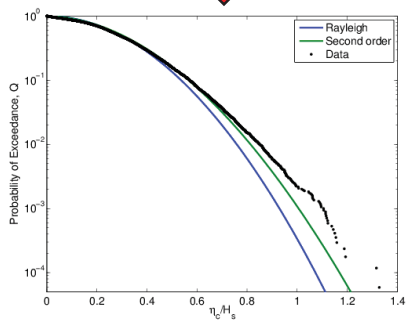
- Directionally spread, focused wave events; σ_θ=15°
- Most probable shape of large linear waves: Lindgren (1970)
- Significant amplification beyond second order
- Confirmation of laboratory data



(iv) Comparisons between laboratory & field data



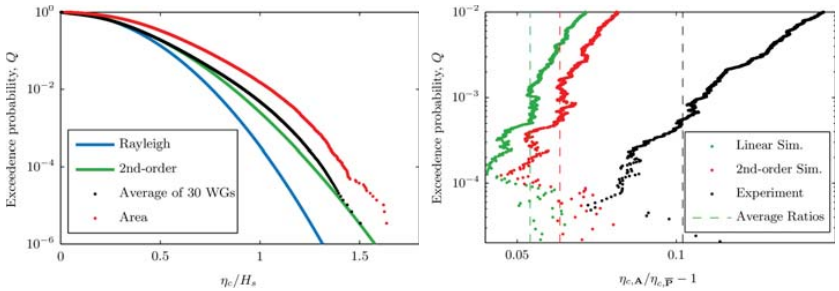
Laboratory data H_s=12.5m, σ_θ=15°
Latheef & Swan, Proc. Roy. Soc. A (2013)



Field data (H_s>12m):
Christou & Ewans (2012), *CREST* JIP

Area effects

Comparisons between point-averaged and area maximum statistics

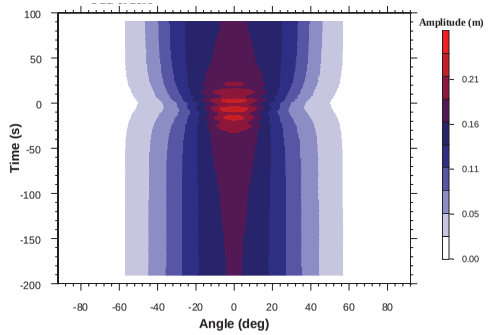


- Spatial effects > linear & 2nd-order predictions
- Nonlinear changes in spectral bandwidth are key
- Largest waves are more narrow-banded (even in a linear sense)
- Nonlinear persistence becomes important
- For $Q \approx 10^{-3}$, area increase is 10% (on top of nonlinear amplification)

Spatial shape: directionality

Earlier work:

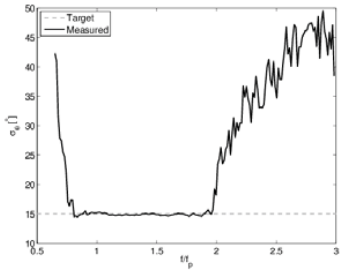
- Numerical calculations of focused waves (spectral model - BST)
- Local reduction in directional spreading
- Supporting laboratory data (Johannessen & Swan, 2001 & 2003)



Directionality: nonlinear changes

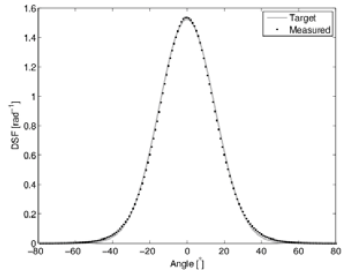
Comparisons to laboratory data (H_s=10m, ½$H_s k_p$=0.081, $k_p d$=2.0)

- σ_θ=15°
- input data: η,u,v
- calculated using the EMEP
- sea state generated using RDM



σ_θ vs. f/f_p

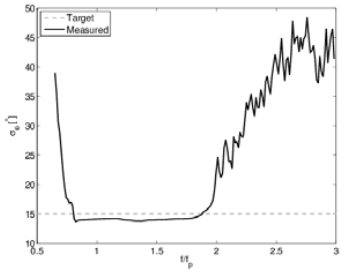
Directional spreading function, *DSF*



Directionality: nonlinear changes

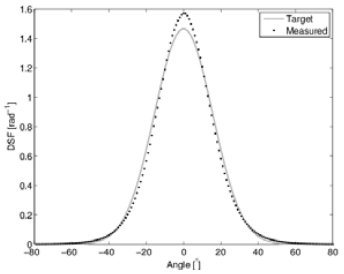
Comparisons to laboratory data (H_s=15.0m, ½$H_s k_p$=0.122, $k_p d$=2.0)

- σ_θ=15°
- input data: η,u,v
- calculated using the EMEP
- sea state generated using RDM



σ_θ vs. f/f_p

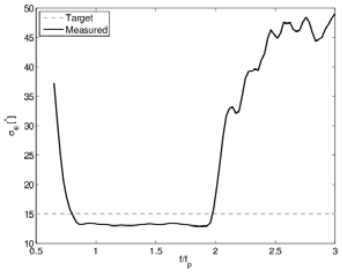
Directional spreading function, *DSF*



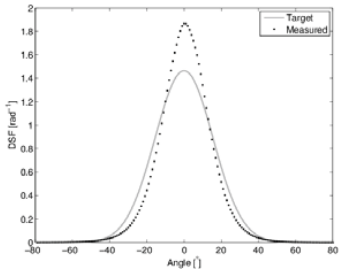
Directionality: nonlinear changes

Comparisons to laboratory data (H_s=20.0m, ½$H_s k_p$=0.163, $k_p d$=2.0)

- σ_θ=15°
- input data: η,u,v
- calculated using the EMEP
- sea state generated using RDM



σ_θ vs. f/f_p

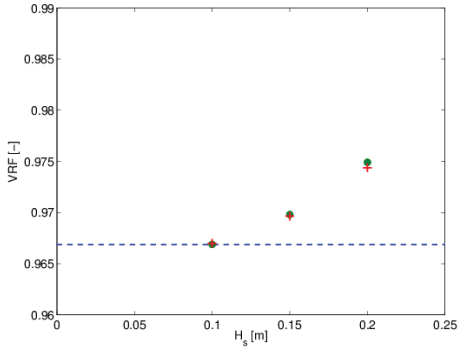


Directional spreading function, *DSF*

Conclusion: steepest seas are more uni-directional

Velocity Reduction Factor (VRF)

- Comparisons to laboratory data
- VRF averaged over 20 x 3-hour seeds for each sea state
- Sea state averaged values; changes with H_s

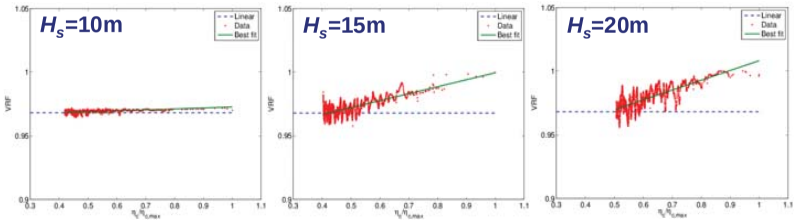


- - - linear input
● velocity ratio
+ earlier EMEP

Conclusion: steepest seas are more uni-directional

Velocity Reduction Factor (VRF)

- Comparisons to laboratory data (H_s=10m, σ_θ=15°, ½$H_s k_p$=0.081)
- VRF calculated for individual waves, based on lab data (σ_θ=15°)
- Plotted in terms of the normalised crest elevation, η_c/η_{cmax}

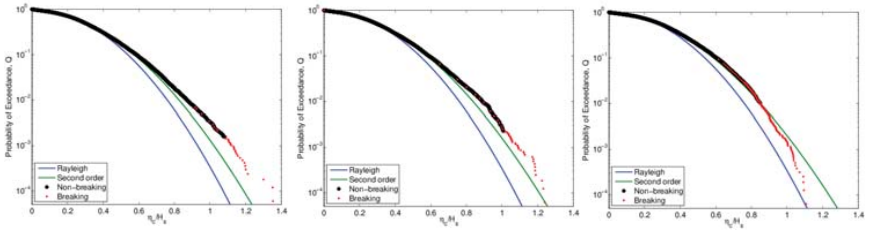


- **Conclusion: largest waves are more uni-directional in steep seas**
- **Does not mean *UD* model testing is appropriate (or conservative)**

The occurrence of wave breaking

SHORTCREST JIP:

- Visual observations allow breaking waves to be identified
- Where breaking is dominant (on average) data is given in red
- With increasing steepness, the tail of the distribution is controlled by breaking, hence the reduction in crest heights
- Laboratory data relates to σ_θ=15°



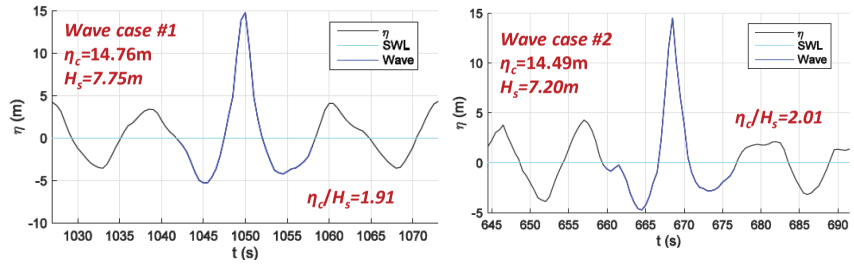
H_s=15m, $H_s k_p$=0.122, σ_θ=15°

H_s=17.5m, ½$H_s k_p$=0.142, σ_θ=15°

H_s=20m, ½$H_s k_p$=0.163, σ_θ=15°

Recent analysis of field data

Date	Crest elevation, η_c (m)	Significant wave height, H_s (m)	η_c / H_s	Wave steepness S_1
2014-01-20	14.76	7.75	1.91	0.059
2014-01-20	14.49	7.20	2.01	0.056
2011-12-08	11.80	6.83	1.73	0.058
2014-01-16	11.41	6.78	1.68	0.050
2011-10-24	10.56	5.93	1.78	0.056



These waves are so steep they will be breaking

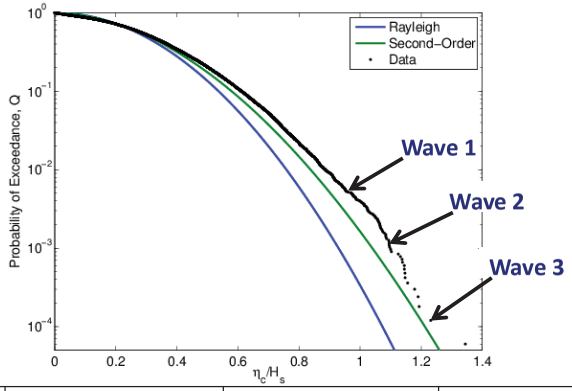
Crest kinematics - measured & predicted

Purpose:

- To identify a number of representative extreme waves
- To contrast the measured data with commonly applied design wave solutions
- To assess the role of:
  - Nonlinear amplification
  - Wave breaking
- All of the data corresponds to North Sea conditions lying on the 10^{-4} (H_s, T_p) environmental contour

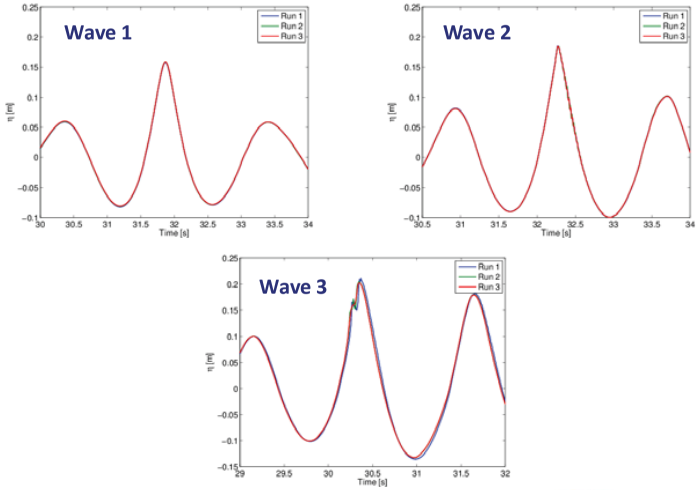
H_s	T_p	d	$S_{\eta\eta}(\omega)$	σ_θ
17.2m	16.0s	93m	JONSWAP	Ewans

Three selected wave events

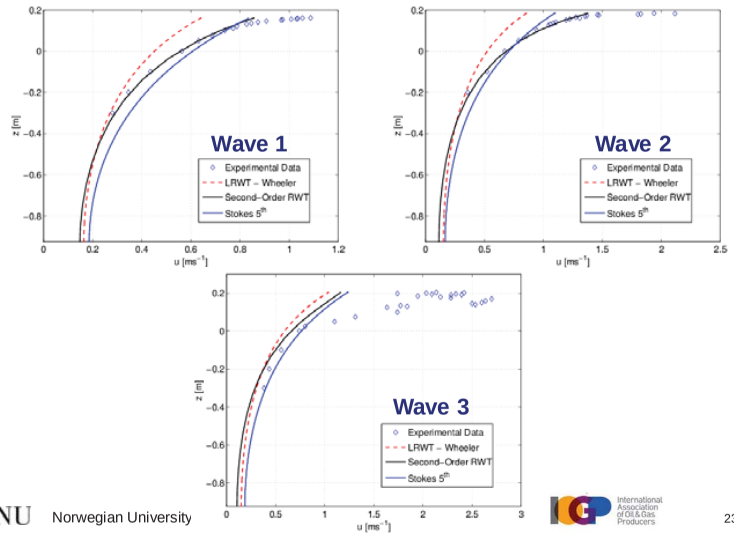


Wave case	description	crest elevation, η_c (m)	local period, T_{local} (s)
1	non-breaking	16.2	14.0
2	spilling	18.6	12.8
3	over-turning	20.9	12.9

Repeatability of wave records:



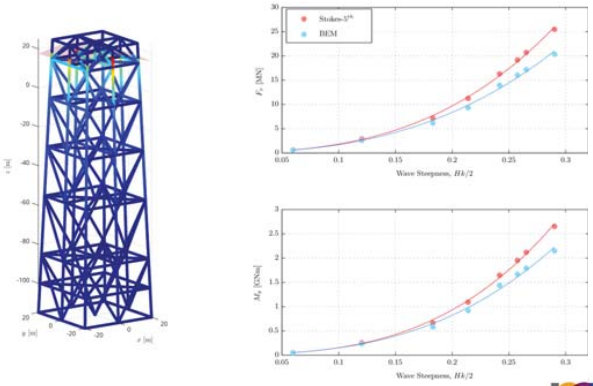
Recorded kinematics, $u(z)$ beneath η_{max}



Practical implications

(i) **Sub-structure loads**

- Calculations based on regular waves will be conservative
- 15-20% reduction for 40m x 40m base area in steep waves
- Important opportunity when it comes to re-assessment



Practical implications

(ii) **Wave-in-deck (WID) loads**

Preferred method of calculation

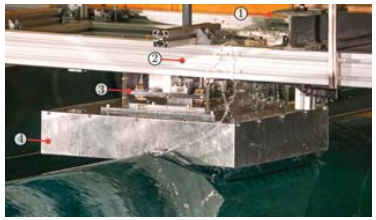
- Based on a momentum exchange (from wave to structure)
- Physically realistic
- Relatively easy to apply

Force = rate of change of momentum
∝ mass flow rate x velocity
∝ $\rho \Delta\eta u^2$ where $\Delta\eta$ = deck inundation

- WID loads dependent upon (a) **Fluid velocity** (strongly)
  (b) Volume flux
  (c) Inundation
  } **Wave shape**

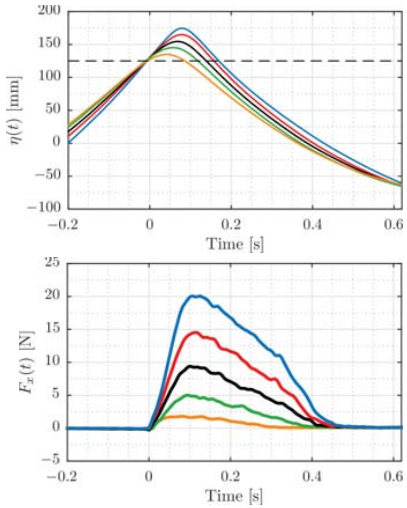
WID loads: laboratory observations

State-of-the-art wave basin



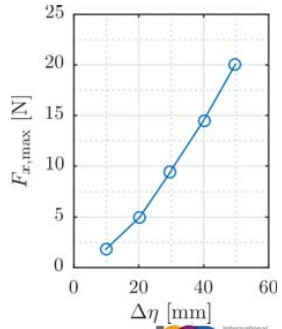
Generic topside & 6-axis load cell

WID loads: variation with inundation

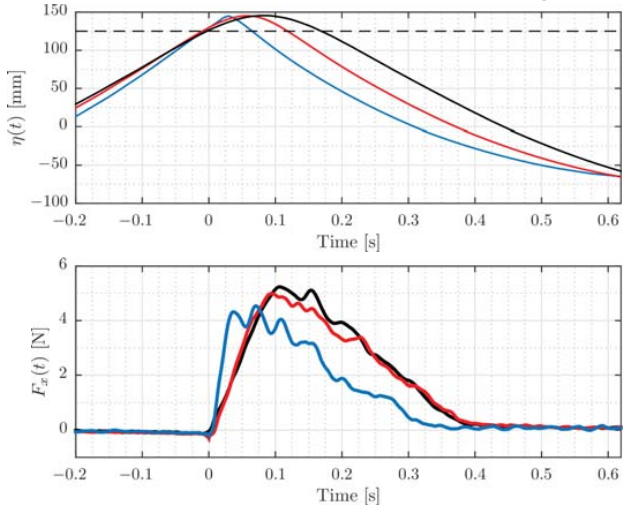


Jonswap, T_p=1.6s, γ=2.5, h_d=125mm, non-breaking, with $\Delta\eta$=10mm, 20mm, 30mm, 40mm and 50mm

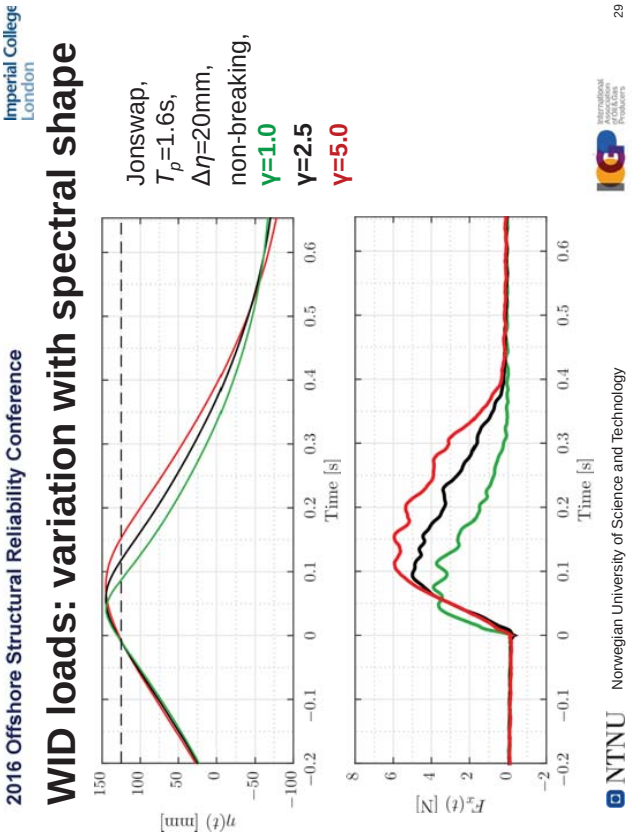
(All dimensions at laboratory scale)



WID loads: variation with T_p



Jonswap,
$\Delta\eta$=20mm
γ=2.5
non-breaking,
T_p=1.8s
T_p=1.6s
T_p=1.4s


2016 Offshore Structural Reliability Conference
Imperial College London
WID loads: variation with spectral shape
Jonswap,
T_p=1.6s,
$\Delta\eta$=20mm,
non-breaking,
γ=1.0
γ=2.5
γ=5.0
$\eta(t)$ [mm]
$F_x(t)$ [N]
Time [s]
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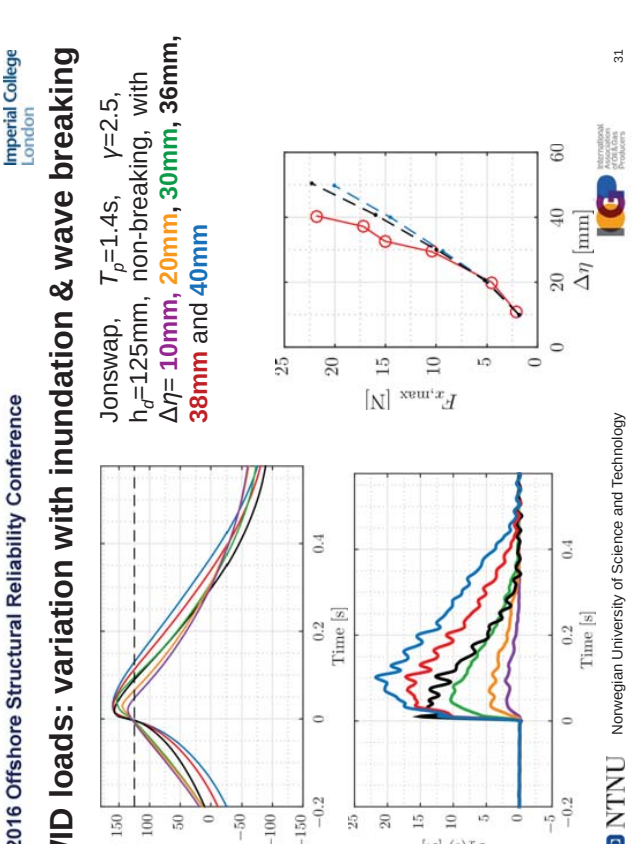
29


2016 Offshore Structural Reliability Conference
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WID loads: variation with inundation & wave breaking
Jonswap, T_p=1.4s, γ=2.5,
h_d=125mm, non-breaking, with
$\Delta\eta$= 10mm, 20mm, 30mm, 36mm,
38mm and 40mm
$\eta(t)$ [mm]
$F_x(t)$ [N]
Time [s]
$F_{x,max}$ [N]
$\Delta\eta$ [mm]
NTNU Norwegian University of Science and Technology
31

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WID loads: variation with spectral shape
Jonswap,
T_p=1.6s,
$\Delta\eta$=20mm
non-breaking,
γ=1.0
γ=2.5
γ=5.0
Regular wave
$\eta(t)$ [mm]
$F_x(t)$ [N]
Time [s]
NTNU Norwegian University of Science and Technology
30

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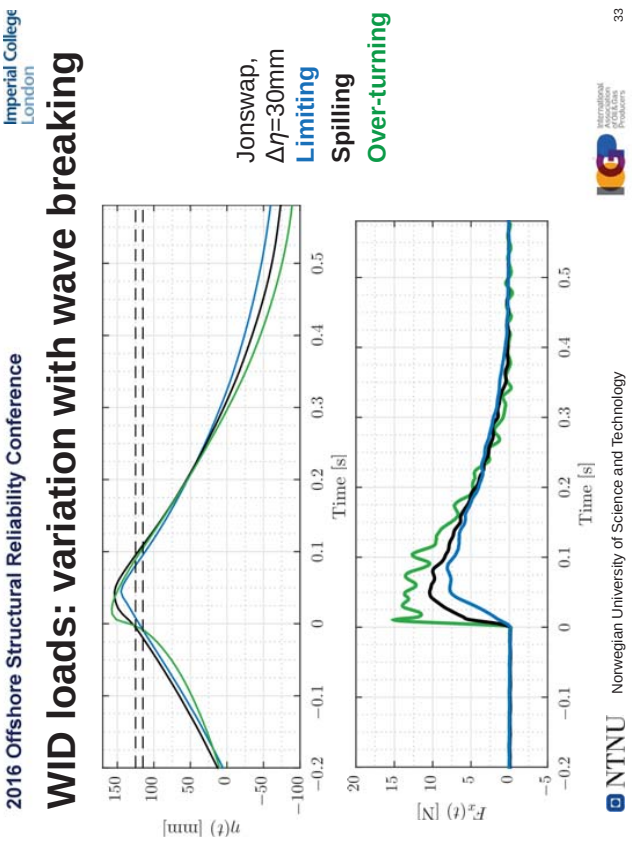
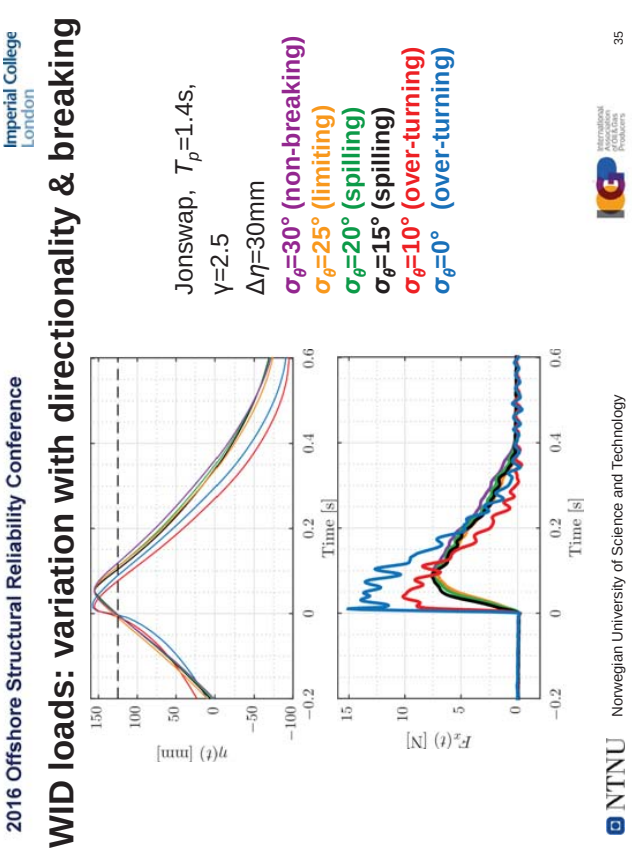
WID loads
Wave shape
(A)
(B)
(C)
(D)
(A) Non-breaking, (B) limiting, (C) spilling and (D) over-turning
NTNU Norwegian University of Science and Technology
32


2016 Offshore Structural Reliability Conference
Imperial College London
WID loads: variation with wave breaking
Jonswap, Δη=30mm
Limiting
Spilling
Over-turning
η(t) [mm]
Fx(t) [N]
Time [s]
NTNU Norwegian University of Science and Technology
33


2016 Offshore Structural Reliability Conference
Imperial College London
WID loads: variation with directionality & breaking
Jonswap, Tp=1.4s,
γ=2.5
Δη=30mm
σθ=30° (non-breaking)
σθ=25° (limiting)
σθ=20° (spilling)
σθ=15° (spilling)
σθ=10° (over-turning)
σθ=0° (over-turning)
η(t) [mm]
Fx(t) [N]
Time [s]
NTNU Norwegian University of Science and Technology
35

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WID loads: wave kinematics (BEM calculations)
(A) Non-breaking, umax=0.92m/s
(B) Over-turning, umax=1.59m/s
z [mm]
x [mm]
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2016 Offshore Structural Reliability Conference
Imperial College London
WID loads: variation with spectral shape & breaking
Jonswap, Tp=1.4s,
γ=2.5
Δη=20mm
γ=1.0 (over-turning)
γ=2.5 (limiting)
γ=5.0
Regular wave
η(t) [mm]
Fx(t) [N]
Time [s]
NTNU Norwegian University of Science and Technology
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Concluding remarks

(1) Wave crest statistics

- Effects beyond 2nd-order can be significant
- Competing influence of:
  - nonlinear amplification (3rd-order resonant effects)
  - energy dissipation due to wave breaking

(2) Wave breaking

- Key factor influencing extreme load predictions
- Governed by local steepness, including directional effects
- Should be viewed as a process
  limiting ➡ spilling ➡ over-turning
- Rapid changes in the local water particle kinematics

Concluding remarks (continued)

(3) Load predictions

- Critically dependent on wave kinematics
- Comparisons with regular wave predictions are poor
  - sub-structure loads will be conservative (over-predicted)
  - wave-in-deck loads will differ significantly
    - volume of water entering the deck becomes an important factor
- It should not be assumed that $F_{WID} \propto \Delta\eta$
- Smaller breaking waves can produce larger loads
- Requires search across the relevant H_s-T_p domain to identify worse case loading events
- New procedures are being developed within the *LOADS JIP* to introduce new physical understanding within an efficient Monte Carlo simulation.

Questions?

The 3rd Offshore Structural Reliability Conference
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14-16 September, Stavanger, Norway

The loading and reliability of fixed steel structures in extreme seas: recent advances and required improvements

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ABSTRACT: This paper concerns the calculation of the fluid loads acting on a steel jacket structure, as required for a structural reliability assessment. It addresses both the sub-structure loads and the wave-in-deck loads, highlighting the practical implications of recent advances in the description of extreme ocean waves. These advances include the nonlinear amplification of crest elevations beyond second-order, the occurrence of wave breaking (both spilling and over-turning), the role of directionality (particularly in relation to the highest and steepest individual waves), the local and rapid spatial evolution, and the description of the water particle kinematics high in the wave crest. Taken together, these changes ensure that the load statistics will be very different to those based on linear or second-order theory. Moreover, the maximum loads relating to small exceedence probabilities will differ from deterministic calculations based on un-physical regular wave theories; the fifth-order Stokes solution being commonly adopted in present practice. Calculations undertaken to date suggest that present practice is likely to be conservative in respect of sub-structure loads, but non-conservative in terms of wave-in-deck loads; the latter potentially becoming very significant when large levels of wave inundation are concerned. In addressing these issues, recommendations are made concerning the implementation of improved modelling approaches; the ultimate goal being to achieve a physically realistic solution (including the effects of wave breaking) without the need for excessive laboratory or numerical calculations.

KEY WORDS: Extreme waves; nonlinear crest heights; wave breaking; wave-in-deck loads; sub-structure loads

1 INTRODUCTION

An accurate determination of the reliability of an offshore structure is a crucial aspect of both the initial design of a new structure and the re-assessment of an existing structure. In simple terms it involves assessing the total applied load (particularly the environmental loads) and the structural resistance; determining the return period at which the former exceeds the latter. In essence it reduces to a loads *vs.* resistance problem; the present state-of-the-art being one in which the vast majority of the uncertainty resides on the loading side. Indeed, if an improved estimate of the structural reliability is to be achieved, the present authors believing that this is both possible and critically important, then it will be driven by a better understanding of the environmental loads, particularly the extreme loads produced by the largest and/or steepest waves and wave-current combinations.

In addressing the applied loads, consideration needs to be given to both the sub-structure loads and the occurrence of wave-in-deck (WID) loading. It is in respect of the latter that an initial design and a re-assessment begin to differ; the magnitude (and uncertainty) of any WID loads being such that a new structure is designed to avoid their occurrence, even in the most extreme waves. This is achieved by setting a deck elevation that is sufficiently high to maintain a positive air-gap over the proposed life of the structure. Unfortunately, such an approach cannot easily be adopted in the case of an existing structure. Moreover, it has become clear from recent experience, both in terms of structural failure and the results of reliability studies, that when significant WID loading arises it can have a profound effect on the survivability of a structure. To understand the importance of WID loading, the following points need to be recognised:

(1) With improved hydro-carbon recovery, there is a desire to extend the life of many existing structures.
(2) The design criteria appropriate to these older structures has been superseded. This has occurred at several levels including:
    (a) A small but upward drift in the design met-ocean criteria, particularly H_s.
    (b) The increasing desire, by the regulatory authorities, to impose a 10^{-4} rather than a 10^{-2} annual exceedance probability.
    (c) An improved understanding of extremes, including both the long-term sea state parameters and the short-term wave characteristics.
(3) The desire to operate platforms with a larger topside load.
(4) Reduced air gaps (for a given crest elevation) due to a combination of sea level rises, increased storm surge and, in some cases, substantial sea bed settlement due to reservoir depressurisation.

When taken together these points ensure that the effective determination of WID loading represents an important part of many re-assessments. The present paper will assess how recent scientific advances, many developed within the recent *CREST* and *SHORTCREST* Joint Industry Projects (JIP's), contribute to this process. In accordance with the uncertainties noted above, the paper will concentrate on the loading side of

the equation, providing inputs appropriate to both the sub-structure and the WID loads.

2 BACKGROUND

When considering the total wave loads, the component acting on the sub-structure will be drag dominated assuming the Keulegan-Carpenter number is defined by $KC=UT/D>20$ at z=0; where u is the wave-induced horizontal velocity, T the wave period and D the diameter of the individual member under consideration. In contrast, the WID loading is best calculated on the basis of a momentum exchange; the momentum of the water entering the deck being progressively destroyed, the rate of destruction depending on the porosity of the deck structure. Taken as a whole, the applied wave load will increase with the incident crest elevation, but will be 'badly behaving' in the sense that the rate of increase will be substantially larger once the air-gap is reduced to zero and water enters the deck. This emphasises the importance of the WID loading component, its contribution to the total load and its impact on the calculated reliability.

The determination of any WID loads is critically dependent upon:

(i) The incident crest elevation, since this determines the degree of inundation or topside submergence.
(ii) The shape of the wave crest; determining (in part) the total volume of water entering the topside.
(iii) The water particle kinematics high in the wave crest; defining the relevant momentum flux.

Unfortunately, these properties of an extreme wave are among the most difficult to define. The reason for this is that the largest, most extreme waves will be steep and therefore highly nonlinear. The origins of this nonlinearity lie in the kinematic and dynamic boundary conditions applied on the water surface; the effects of the nonlinearity growing with proximity to the water surface, particularly high in the wave crest. Moreover, if the wave steepness is such that the wave begins to break, as will often be the case in the largest waves in the most severe sea states (see Section 3.2 below), this will have a profound effect on (i), (ii) and (ii) above; the change in (iii) primarily influencing that part of the wave crest that determines the WID loading.

In calculating the WID loads, the transfer of momentum (from the wave to the structure) is given by the product of the mass flow rate of water entering the deck and its associated velocity. Since the mass flow rate is itself proportional to the water inundation and fluid velocity, the applied force is defined by

$$F_{WID}\ \alpha\ \Delta\eta \times [U(z\approx\eta_c)]^2 \tag{1}$$

where $\Delta\eta$ is the deck inundation and $U(z\approx\eta_c)$ is the horizontal velocity high in the wave crest, close to the instantaneous water surface $z\approx\eta_c$. It is clear from (1) that if the wave nonlinearity (or, more significantly, the occurrence of wave breaking) leads to a change in $U(z\approx\eta_c)$, the corresponding change in F_{WID} will be two-fold.

In making this argument, the loads have been assumed to be dependent on the momentum flux. At this stage it is important to note that other approaches assume a drag-type loading description. The present authors do not subscribe to this view, not least because a Morison's loading formulation assumes the loads can be based upon the incident undistributed fluid velocity. In the case of WID loading the fluid is brought to rest very rapidly, particularly in the case of a densely packed or plated structure. As a result, the application of an undistributed slender body theory is difficult to justify, introducing large uncertainty in the appropriate loading coefficients. However, setting aside these concerns, a drag-type loading calculation also assumes the WID loads are proportional to $[U(z\approx\eta_c)]^2$. This ensures that any change in the water particle kinematics again leads to a two-fold change in F_{WID}.

Taking a broader view, wave nonlinearity will alter the water particle kinematics throughout the entire water column. As a result, it also has implications for the sub-structure loads. In particular, if the relevant failure mode is over-turning about the base, moment-arm effects will further emphasis the largest loads arising high in the water column and hence the area in which wave nonlinearity has its largest effect.

It is clear from these comments that whilst the sub-structure loads are sensitive to nonlinear changes in the predicted wave kinematics, the WID loading is entirely dependent on that part of the wave field that is subject to the greatest uncertainty and least well described by the commonly adopted wave solutions.

3 WAVE MODELLING – RECENT ADVANCES

It has already been noted that the prediction of the wave loads, particularly the WID loads, is critically dependent upon the wave crest elevations, η_c, and the underlying water particle kinematics, (u,v,w) in the (x,y,z) directions; where z is measured vertically upwards from the mean water level and x is aligned with the mean wave direction. Both η_c and (u,v,w) must accurately describe the largest, and perhaps also the steepest, wave events arising in a given return period. Recent research has shown that if these descriptions are to be accurate they must incorporate the underlying physics of the sea states within which the individual waves arise. Specifically, this should include the unsteadiness, the nonlinearity and the directional spread; the three properties being inter-dependent. In practice, the largest waves will exhibit marked departures from expected linear or second-order behaviour. This will include high-order nonlinear amplifications, the dissipative effects of wave breaking, nonlinear changes in directionality, rapid spatial evolution and extreme water particle kinemics. Since each of these characteristics can have a profound influence on the applied fluid loads, and hence the results of any reliability analysis, they will be addressed individually.

3.1 Nonlinear amplification beyond second-order

Assuming a sea state is linear and narrow banded, the crest heights will be Rayleigh distributed. In an attempt to incorporate some nonlinearity, the present state-of-the-art in terms of design practice is based upon [1]. This defines a two-

parameter Weibull fit to the crest elevations predicted using the second-order random wave solution described in [2]; the latter including the frequency-sum and frequency-difference terms first identified in [3]. The convenience of the model outlined in [1] lies in the fact that it provides second-order accurate crest height statistics for a range of sea state steepness's and effective water depths without having to undertake long time domain simulations. However, it is limited in the sense that it only includes terms up to second-order. Specifically, it neglects the third-order *resonant* and *near-resonant* terms that have been shown to produce a rapid and spatially localised amplification of the steepest and largest wave crests [4]. Confirmation of the importance of these terms in realistic directionally spread JONSWAP spectra were provided in the *CREST* and *SHORTCREST* JIP's and reported in [5]. This evidence was based on long time domain simulations of random waves in a laboratory wave basin; the results confirmed by independent tests undertaken in a second laboratory wave basin.

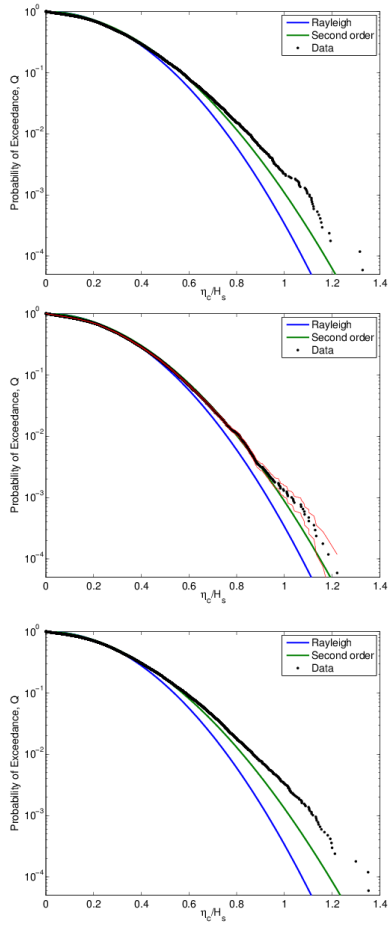

Figure 1. Crest height distributions recorded in a laboratory wave basin, T_p=16.0s, σ_θ=15°and (a) H_s=10.0m, (b) H_s=12.5m and (c) H_s=15.0m.

Examples of this data are given in Figure 1; sub-plots (a)-(c) providing normalised crest height distributions (η_c/H_s, where H_s is the significant wave height) for three sea states of increasing steepness. In each case comparisons are made between the measured data and predictions based upon the linear Rayleigh distribution and the second-order Forristall model. It is clear from these results that significant amplifications beyond second-order are observed and that the magnitude of these amplifications increases with the sea state steepness. Indeed, it is clear that the differences between the fully nonlinear laboratory observations and the second-order predictions are at least as large as the difference between linear and second-order; the absolute magnitude of the amplification being largest in the tail of the distribution.

Further consideration of the nonlinear amplification of crest heights beyond second-order has been provided by the analysis of field data [6,7], also undertaken within the *CREST* JIP. This considered a very large data base, applying a rigorous quality control procedure. If all deep water records with H_s>12 m are considered, the assembled crest height distribution (again normalised with respect to H_s) is given in Figure 2. In this case, comparisons to the linear (Rayleigh) and second-order (Forristall) distributions again show significant nonlinear amplifications. Indeed, these results are strikingly similar to the equivalent laboratory observations given on Figures 1(b) and 1(c). Whilst the results presented on Figures 1 and 2 are new, they were not unexpected. Real seas, particularly those relevant to design conditions, are not second-order.

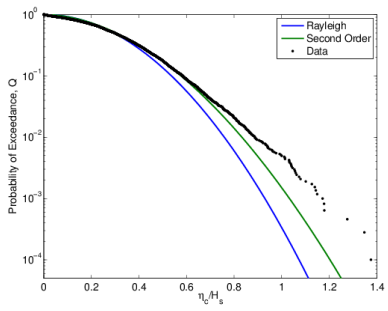

Figure 2. Crest height observations in the field (H_s≥12m); data taken from [7].

3.2 *The dissipative effects of wave breaking*

Whilst the sea states considered in Figures 1 and 2 are large, being representative of the 100-year or 1000-year design conditions for the North Sea, present design practice requires larger (10,000-year) sea states to be considered. Figure 3 provides two further examples of laboratory generated sea states. Figure 3(a) relates to H_s=17.5 m and Figure 3(b) H_s=20.0 m; the former being representative of 10,000-year conditions in the North Sea and the latter tropical cyclone conditions. In both cases it is clear that whilst the nonlinear amplifications persist, the tail of the distribution falls back towards the second-order solution. Indeed, in the steepest sea state it falls well below second-order, approaching the linear Rayleigh distribution. This reduction arises due to the limiting and dissipative effects of wave breaking; the latter being governed by the steepness of the individual wave events.

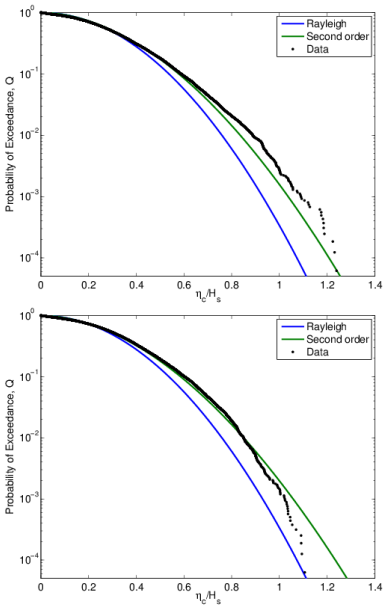

Figure 3. Crest height distributions showing the dissipative effects of wave breaking, T_p=16.0s, σ_θ=15°and (a) H_s=17.5m and (b) H_s=20.0m.

In addition to the long time-histories of $\eta(t)$ recorded in the wave basin, simultaneous video records allowed us to establish the occurrence of wave breaking. Figures 4(a)-(b) reconsiders the crest height distributions relating to the two steepest sea states and identifies (coloured in red) those individual wave events that were observed to be breaking, either spilling or over-turning. It is clear from these results that many of the largest wave events involve some degree of wave breaking; the correlation between that part of the distribution that exhibits reduced crest heights and the occurrence of breaking being clearly made. These results have two important implications. First, the reduction in the crest heights for the smallest exceedance probabilities may limit the occurrence of WID loads; the onset of wave over-turning providing an effective upper-bound for the crest elevation given a local wave period. Second, when wave breaking occurs, particularly wave over-turning, the fluid velocities high in the wave crest will be dramatically increased (see Section 3.5 below).

3.3 Nonlinear changes in the directional spread

Unless they are entirely dependent upon swell waves, all design sea states will be directionally spread. With individual wave components travelling in different directions, the wave-induced fluid velocity in the mean wave direction will be reduced. Indeed, the so-called *velocity reduction factor* (expressing the ratio of the velocities in a directionally spread sea to the equivalent velocities in a uni-directional sea) is often used as a measure of directionality. Expressed in this form it is easy to conclude that with the introduction of a progressively larger directional spread, the fluid velocities and hence the corresponding fluid loads will reduce. In effect, the neglect of directionality is entirely conservative.

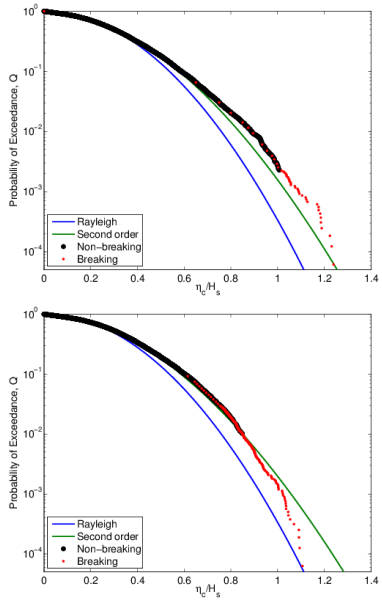

Figure 4. Crest height distributions highlighting individual wave events (in red) observed to be breaking (by both spilling and over-turning): T_p=16.0s, σ_θ=15°and (a) H_s=17.5m and (b) H_s=20.0m.

Unfortunately, the fact that directionality has a direct (linear) influence on the steepness of an individual wave event ensures that the role of directionality is more complicated. For example, with the onset of wave breaking critically dependent on the inline wave front steepness, the introduction of a directional spread sea will lead to the reduced (or delayed) occurrence of wave breaking. As such, large individual waves may exhibit higher crest elevations in directionally spread seas. Moreover, setting aside the occurrence of wave breaking, the directionality of both the sea state as a whole and of large individual waves within a given sea state will be dependent on the nonlinearity. Evidence of this is provided on Figures 5 and 6 respectively. Figures 5(a)-(c) concern three sea states with increasing steepness; H_s=10m, 15m and 20m, with T_p held constant at 16s. In all cases the target (linear) directional spread was normally distributed with a standard deviation of σ_θ=15°. In Figure 5(a), corresponding to H_s=10m, the agreement between the measured data and the target directional spread is extremely good. However, in Figures 5(b) and (c), an increase in the nonlinearity of the sea state leads to a reduction in the directional spread. Whilst acknowledging that the accurate determination of a directional spread is difficult, the present results are based upon the extended maximum entropy principle [8] using coincident time-histories of the water surface elevation and the two horizontal velocity components (η,u,v). Full details of this approach are given in [9], together with all the necessary data assurance. The clear conclusion arising from these results is that the directionality of a sea state as a whole reduces with an increase in the sea state steepness.

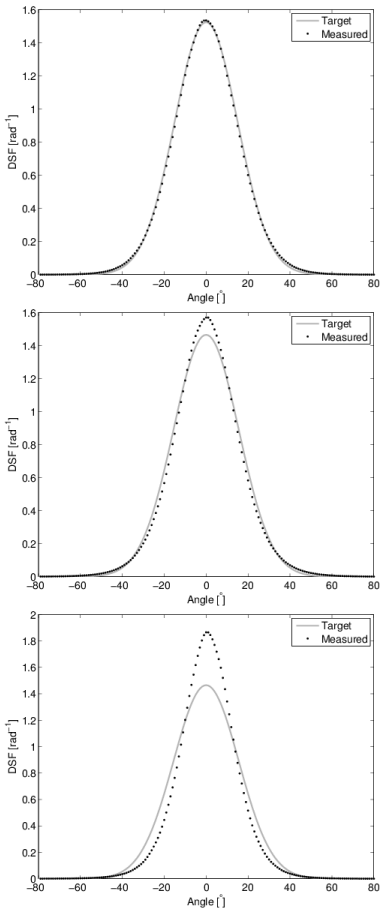

Figure 5. Nonlinear changes in the directional spreading function; T_p=16.0s and (a) H_s=10.0m, (b) H_s=15.0m and (c) H_s=20.0m.

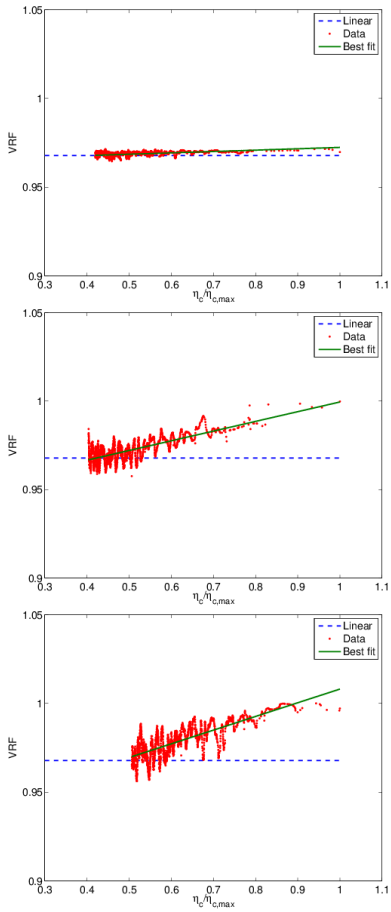

Figure 6. The directionality of large individual wave events (based on the velocity reduction factor) in sea states defined by T_p=16.0s and (a) H_s=10.0m, (b) H_s=15.0m and (c) H_s=20.0m.

In contrast, Figures 6(a)-(c) concern the directionality of the largest individual waves occurring in each of the three sea states considered previously. Since the spreading parameter σ_θ is only appropriate to the sea state as a whole, the directionality of individual wave events is based upon the velocity reduction factor. These results show that with an increase in the sea state steepness, the largest waves become more uni-directional; the velocity reduction factor tending to 1.0. This is consistent with earlier numerical calculations of deterministic focused and near-focused waves [4,10] and anecdotal evidence involving visual observations of very long-crested "walls of water".

Having established that both the sea state as a whole and, particularly, large individual waves within that sea state become less directionally spread as the sea state steepness increases, it must be emphasised that these represent real nonlinear changes. As such, they need to be incorporated within our characterisation of a design sea state, but do not imply that model testing should be undertaken in uni-directional seas.

3.4 Rapid spatial evolution

The data presented in Figures 1-6 all relate to data gathered at a single point. In practice, jacket structures and particularly the topside structures have a finite plan area. Whilst it is well known that area statistics will differ from point statistics, much of this work has been based on linear or second-order calculations. Figure 7 concerns data recorded over an equivalent full-scale plan area of 60m x 60m. Figure 7(a) contrasts the average of the point statistics ($\eta_{cP}(Q)$) recorded at 49 (7x7) equi-spaced gauges located on a 10m x 10m grid with the area maximum ($\eta_{cA}(Q)$); the latter identified using a variable time interval such that the maximum defines the larger crest elevation corresponding to the propagation of a single wave event over the plan area. The first point to note is that the area maxima are substantially larger than the point-averaged statistics.

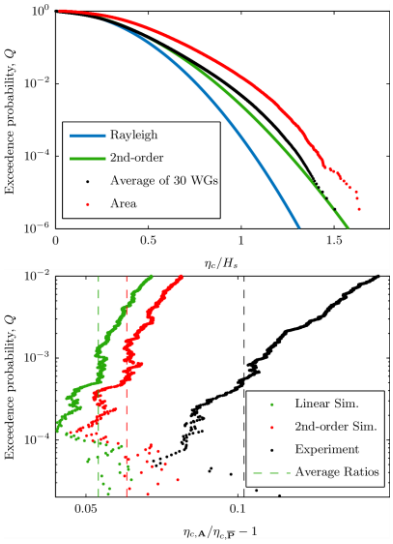

Figure 7. Spatial effects; (a) a comparison between point and area statistics and (b) the percentage increase in crest heights with comparisons between linear, second-order and fully nonlinear laboratory data.

Using the data given on Figure 7(a), Figure 7(b) defines the fractional increase, $((\eta_{c\mathrm{A}})/(\eta_{c\overline{\mathrm{P}}})-1)$, for a given exceedance probability. This contrasts the measure data with equivalent linear and second-order calculations based on long time domain simulations at the same number of spatial locations. These comparisons identify two important trends. First, the measured data exhibits a larger area effect when compared to both the linear and second-order calculations. Second, whilst all three data sets exhibit a smaller area effect with reducing exceedance probabilities, the measured data shows the greatest rate of reduction.

The explanation for these observations lies in three parts.

(a) The increased area effect present in the laboratory data is in large part due to the nonlinear wave-wave interactions. In intermediate and deep water the third-order *resonant* terms produce a local shifting of wave energy into the higher harmonics. This leads to an effective broadening of the freely propagating wave spectrum and hence an increase in the area effect. It also has implications for the water particle kinematics which will be considered in Section 3.5.

(b) In a linear sense a large wave is caused by the focusing of freely propagating wave components. Whilst nonlinear effects may lead to larger waves, and may also alter the details of the focusing process, in intermediate and deep water the combination of frequency and directional focusing remains the driving mechanism. In a linear sense it has been shown by [11, 12, 13 and 14], that the largest waves are based on the summation of component amplitudes that are proportional to the wave spectrum rather than the square root of the spectrum. This implies a narrowing of the spectral bandwidth such that the largest waves are less dispersive and therefore exhibit reduced area effects. This explains the general reduction in the area ratio for smaller exceedance probabilities (or larger crest elevations) irrespective of the order of the calculations involved.

(c) The increased gradient of the area ratio based on the laboratory data is believed to be due to nonlinear (spatial) persistence. When a large wave forms, the full nonlinearity of the problem leads to a more rapid focusing (linked to point (a) above), but a more gradual de-focusing. In effect, the very largest waves are more persistent. This is particularly evident when wave breaking occurs. Although this is fundamentally a dissipative process, it also involves a degree of phase locking so that the breaking process extends over a larger spatial area. Whilst this may not be large in comparison to the typical wave length, it will be significant in terms of the plan area of a typical jacket structure; effectively reducing the area effect in the largest most damaging waves.

Table 1. Individual wave properties.

wave case	description	crest elevation, η_c (m)	local period, T_{local} (s)
1	non-breaking	16.2	14.0
2	spilling	18.6	12.8
3	over-turning	20.9	12.9

3.5 *The water particle kinematics*

Having already demonstrated the importance of the nonlinear wave-wave interactions, it is to be expected that they will have an equally important effect on the wave kinematics. Specifically, the transfer of wave energy to the higher-frequency components, responsible for the nonlinear amplification of the crest heights[1], will cause an increase in the near-surface velocities appropriate to WID loading. However, such components will decay rapidly with depth leading to smaller fluid velocities and a possible reduction in the sub-structure loads. To investigate these effects, three intermediate depth wave cases serve as appropriate examples. These were all recorded in a 10^{-4} design sea state appropriate to North Sea conditions. The first example is non-breaking, the second spilling and the third over-turning; the local wave properties being defined in Table 1. The horizontal water particle kinematics arising beneath the individual wave crests are presented in Figures 8(a)-(c) with comparisons to the commonly adopted design solutions. The latter includes a linear random wave theory with Wheeler stretching, the second order predictions of [2] and a Stokes' fifth-order solution [15]; the latter based upon a fit to the measured crest elevation.

[1] beyond second-order

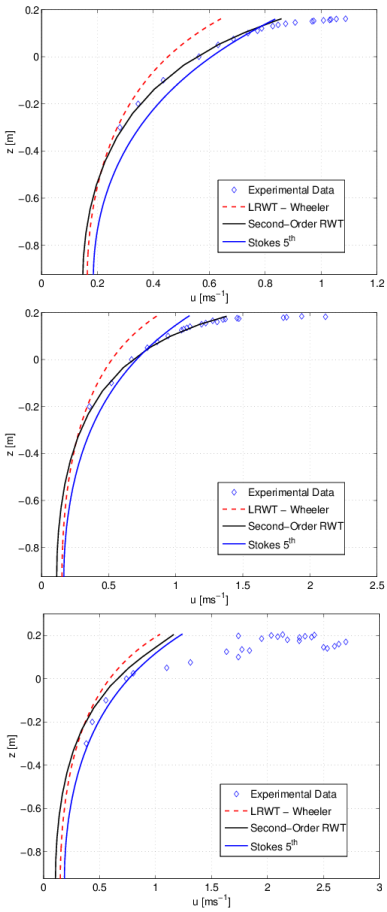

Figure 8. Water particle kinematics recorded beneath three large wave events recorded in a 10^{-4} design sea state; (a) non-breaking, (b) spilling and (c) over-turning (details given in Table 1).

These comparisons show that all the wave models under-predict the largest fluid velocities arising high in the wave crest. In wave cases 1 (non-breaking) and 2 (spilling) this under-prediction is most apparent at the very highest elevations in the wave crest. However, this is the only area that matters for WID loading. In contrast, in wave case 3 (over-turning) the fluid velocities are under-predicted throughout the full depth of the wave crest ($z \geq 0$).

At elevations beneath the SWL, the second-order model works reasonably well. In contrast, the Wheeler solution under-estimates and the Stokes' 5th-order solution over-predicts. These results are to be expected since the waves under consideration (indeed, all waves appropriate to engineering design) are neither linear nor regular. In considering the Stokes' solution it is important to acknowledge that as far as engineering practice is concerned (specifically the description of design waves) it represents a good solution to the wrong problem! This assertion is based on the fact that a uni-directional regular wave solution, however it is applied, cannot possibly incorporate the essential physics governing the properties of a large design wave. Indeed, acknowledging this fact represents the first step in achieving an accurate description of the applied fluids loads and hence the reliability of a given structure.

4 PRACTICAL IMPLICATIONS

This section assesses the implications of the nonlinear changes to the incident wave conditions noted in Section 3. To do this effectively, structures will be grouped into three broad categories depending on the relative importance of any WID loading. In each case suggestions will be made as to how the changes can be effectively introduced within an analysis process.

4.1 Structures maintaining an effective air-gap

If, having reviewed both the long term distribution of storms (giving relevant H_s, T_p and σ_θ values) and the short term distribution of crest heights (the maximum crest elevation typically occurring on the peak of the H_s-T_p environmental contour) the structure maintains an effective air-gap, no WID loading will occur. In this case the nonlinear changes in the incident waves are expected to produce a reduction in the global sub-structure loads when compared to present design practice. The explanation for this lies in the fact that in intermediate and deep water, the nonlinear changes involve a shifting of energy towards the higher frequencies which decay more rapidly with depth. This is equally true of waves that begin to break; the breaking process (both spilling and over-turning), being triggered by the increased near-surface velocities. In these cases the local loads, acting on individual members, high in the water column will increase, but the loads experienced beneath SWL will reduce. The cumulative effect of these changes will depend on the structural form, but the total base shear and the total over-turning moment will typically reduce.

Evidence of this effect is provided on Figure 9. Sub-plot (a) shows a generic jacket structure with a base area of 40 x 40 m while sub-plot (b) concerns the predicted base shear and sub-plot (c) the total over-turning moment. In both Figures 9(b) and 9(c) comparisons are made between calculations based upon a standard 5th-order Stokes solution and a fully nonlinear numerical (boundary element) model; the latter providing a physically accurate description of the wave events. These example calculations suggest a 20% reduction in the base shear and an 18% reduction in the total over-turning moment for a wave steepness consistent with a 10^{-4} wave event.

In discussing related results with leading industrial practioners [16], some surprise was expressed concerning the extent of the nonlinear reductions. Indeed, with the present industry practice (Stokes 5th) calibrated against field data recorded at Shell's Tern platform, it was argued that the over-prediction was likely to be 5-10% at most. Interestingly, this view is not inconsistent with comparable nonlinear calculations. The largest single wave recorded at the Tern platform had a steepness of $\frac{1}{2}Hk$=0.23, with many of the larger waves being of reduced steepness. Taking due account of both the plan area and the layout of this structure, fully nonlinear calculations suggest a reduction of 8% and 6% in the base shear and the over-turning moment. The key point to

note is that the 10^{-4} wave conditions are substantially steeper giving larger load reductions.

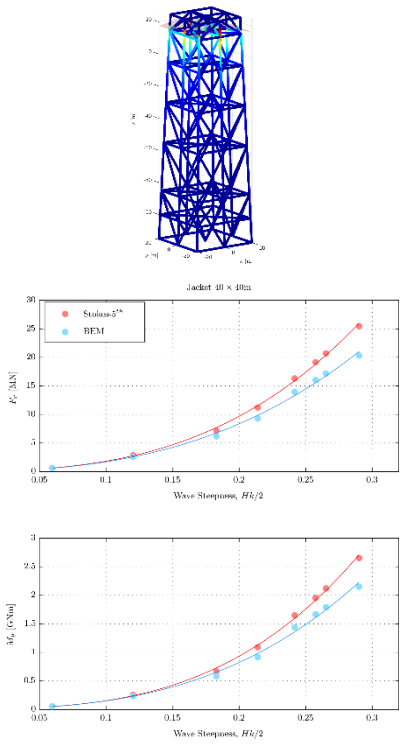

Figure 9. Sub-structure loads: (a) generic jacket structure, (b) predicted base shear and (c) predicted total over-turning moment, with comparisons between Stokes 5th and BEM.

In considering these results, three additional points should be noted

(a) Whilst the global loads reduce, the local loads high in the water column will increase.
(b) The load reductions are based on conditions at the mud-line. For failure mechanisms at higher elevations the results will be quite different.
(c) Given the re-distribution of the applied loads, the strengthening of a structure at high elevations may not have the desired effect.

4.2 Structures encountering small levels of wave inundation

If the deck structure is high but not sufficiently high to ensure that the very largest (10^{-4}) crest elevations do not enter the deck, WID loading will arise and must be carefully assessed. In these cases the nonlinear amplifications (beyond second-order) may be responsible for the unexpected occurrence of wave inundations, but the dissipative effects of wave breaking must not be neglected. This is important on two counts:

(a) In the steepest sea states wave breaking will reduce the very largest crest elevations in the tail of the distributions and may therefore limit the occurrence of WID loading. Indeed, in such cases crest elevations below second-order are consistently recorded. In this regard it is relevant to note that the crest height model outlined in [1] has no imposed breaking limit.
(b) In the highest sea states the largest waves are unlikely to be over-turning. This can be argued on the basis of insufficient wind energy input and hence limits the maximum fluid velocities occurring high in the wave crest.

In such cases it is clear that the fluid velocities high in the wave crest will be under-estimated by a Stokes' solution (Section 3.5). However, the WID loads are also dependent upon the volume of water entering the deck. Since the shape of Stokes' wave is such that this is likely to be over-estimated, some degree of cancellation (between velocity and volume flux effects) is to be expected. As a result, the extent to which the Stokes solution is non-conservative remains unclear; varying from case to case and depending, in part, on the geometry of the structure. In such cases comparisons to a fully nonlinear wave model are required.

In making these comments, we are not arguing that wave breaking does not occur, rather that it is a less significant issue for the very largest waves capable of entering the topside structure. In addressing this category of structures all of the comments noted in Section 4.1 in relation to sub-structure loads remain valid.

4.3 Structures subject to large levels of deck inundation

This represents a small but important sub-group of mostly older structures for which the present results can have very significant implications. Recent studies, relating to specific structures, suggest that present practice, particularly where it is based upon deterministic wave events modelled using a Stokes' solution will be non-conservative; perhaps grossly so. The explanation for this lies in the fact that the WID loading will no longer be proportional to the depth of inundation. Specifically, smaller steeper waves that have the potential to break into the topside structure, particularly where this involves over-turning, may give substantially larger loads when compared to the highest non-breaking crests producing the greatest inundation. With the occurrence of wave breaking controlled by the local wave steepness, the spectral peak period becomes very important, assuming H_s exceeds some threshold at which WID loading events become possible. In the category 2 structures (Section 4.2) the range of T_p values will not be large, but in the present category this is no longer the case.

In the absence of detailed generic models, see below, an experimental determination of the WID loads is preferable. However, given the comments noted above it is not sufficient to merely look along the 10^{-4} H_s-T_p environmental contour, beginning at the peak (maximum H_s) and considering reduced H_s <u>and</u> T_p values; the latter giving steeper sea states. Instead, a wide range of sea states need to be considered throughout the (H_s, T_p) region capable of producing WID loads. Given the importance of wave breaking, the critical loading events may result from a rare wave event in a relatively commonly occurring sea state, rather than the most probable maximum crest height in a rare (10^{-4}) sea. This emphasises the need to

test across the H_s-T_p domain, and to undertake very long random simulations.

For example, a recent analysis of field data suggests that very large waves crests, larger than those traditionally expected (η_c/H_s>2.0), are clearly possible; example data being given in Table 2. Indeed, the fact that they have been recorded, more than once at a single structure, suggests they may not be as rare as initially believed. The obvious explanation for this lies in the nonlinear amplification (Section 3.1) arising in the steepest sea sates.

Table 2. Recent analysis of field data.

Date	Crest elevation, η_c (m)	Significant wave height, H_s (m)	η_c / H_s
2014-01-20	14.76	7.75	1.91
2014-01-20	14.49	7.20	2.01
2011-12-08	11.80	6.83	1.73
2014-01-16	11.41	6.78	1.68
2011-10-24	10.56	5.93	1.78

With a large increase in the magnitude of the predicted WID loads and a reduction in the sub-structure loads, the relative balance between these two loading components will differ from present expectations. This may have implications for the relevant failure modes. Certainly, it should not be assumed that failure occurs by over-turning at the mud-line. Moreover, the relative phasing of the two loading component needs to be addressed. Whilst it is undoubtedly conservative to assume that the maximum WID and the maximum sub-structure loads occur simultaneously, it is equally important to understand that the structural response will be dependent upon the relative phasing of the two loading components. Furthermore, with an appreciation of the increased occurrence of wave breaking (in intermediate and deep water), slam loading will also become more common. This has the potential to act on the topside structure (particularly where this is fully plated), on the supporting deck beams (which are assumed to be part of the topside), and on the upper part (z>0) of the substructure. Whilst the maximum instantaneous loads associated with a wave impact may not be relevant to a static analysis, the total impulse will certainly be relevant to the dynamic response and hence the development of any failure mode.

CONCLUDING REMARKS

The present paper has highlighted a number of important changes in the input parameters appropriate to the calculation of both the sub-structure and the wave-in-deck loads, with specific attention being given to the appropriateness of some presently applied models. The first and perhaps the most immediate conclusion concerns the failure of a deterministic regular wave model (commonly Stokes 5th) to accurately describe the relevant wave properties. As such, these models cannot be recommended and should not be used.

More generally, given the breadth of data necessary to describe the relevant load statistics, particularly for category 3 (Section 4.3) structures, the preferred approach is to develop a number of generic models appropriate to the description of the crest height statistics, the wave shape, the occurrence of breaking and the water particle kinematics. Provided these reflect the full nonlinearity of the problem (without having to undertake fully nonlinear calculations), they can be used as the input to appropriate models describing the sub-structure and the WID loads. If these calculations are undertaken within a Monte Carlo simulation involving all the relevant sea states, the load statistics can be generated. Most importantly, the resulting statistics will incorporate all of the underlying physics noted in Section 3.

This approach is both challenging and very different to commonly adopted practice. Nevertheless, it has recently been successfully undertaken for one specific group of structures [17] and the results shown to be markedly different from earlier deterministic calculation. However, this study involved an enormous effort both in terms of laboratory testing (to determine case-specific models for both the incident wave conditions and the applied fluid loads) and numerical calculations. The challenge for the immediate future is to develop a generic approach, appropriate to a broad range of sea states, water depths and structural forms, and to achieve this without recourse to extensive laboratory testing and with reasonable computational usage. This work is presently on-going within the *LOADS* JIP.

ACKNOWLEDGMENTS

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Chapter 6

Session 4: Reliability Based Calibration of ULS Code Criteria

6.1 Presentation by Marc Maes

NTNU Norwegian University of Science and Technology

International Association of Oil & Gas Producers

Risk-Based Codification for Structural Design and Assessment: Benefits and Challenges

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The University of Calgary
mamaes@ucalgary.ca

The 3rd Offshore Structural Reliability Conference
OSRC2016
14-16 September, Stavanger, Norway

Ponder the following questions:

How is the methodical basis for design standards evolving and expanding?

How does risk in a global interconnected world affect standardization?

Which challenges are we facing using the current generation of design standards?

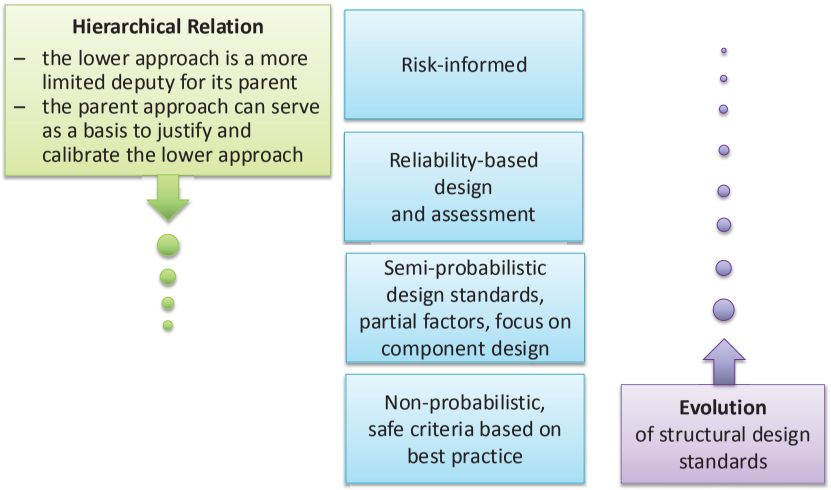
What's new in the 2015 edition of ISO 2394?

- features of the risk-based approach
- the risk-informed constrained optimization
- the basis for life safety risk acceptance

Can a design standard deal with extreme and extraordinary hazards?

What are the key benefits of risk-based standardization?

Methodical basis for design standards



Interconnection and risk

We live in a world of interconnected networks:

- structure and infrastructure networks
- physical, technological, ecological networks
- social, financial networks
- internet, social media, "instant" feedback

In this open world of interconnected networks of everything:

- risk becomes a truly holistic concept
- triggers for risk are ubiquitous in time and in space
- risk can spread very rapidly
- standards that aim to mitigate and control risk in a strictly local or isolated manner become ineffective

Challenges of the current generation of structural design standards

A standard developer's role as the custodian of structural risk and safety conflicts with the unavailability of a solid risk-based codification framework for structural design and assessment

We struggle with how we account quantitatively for any type of non-structural risk. Operational and strategic risk measures cannot be applied directly and transparently for structural design and assessment

Reality check: standards are used increasingly for one-time, unique concepts rather than for repetitive design

It is difficult to harmonize inspection, maintenance planning, asset integrity management, in terms of consistent across-the-board treatment of risk using distinct standards

Standards need to support and promote effective risk communication with both regulators and with the public at large

The 2015 revision of ISO 2394:2015 "General principles on reliability of structures"

The preferred and overarching approach is **risk-informed:**

- the 2015 revision introduces risk and **risk-informed decision making** as the fundamental basis for the standardization of structural safety
- it **integrates** design/assessment with risk-based inspection, maintenance planning, and asset integrity management
- it sets out the required **optimization** and **acceptance** procedure (see slides 8/9)
- risk is **scenario-based** and hinges on the definition of a system (see slide 7)
- the consideration of **robustness** and/or a full system **risk analysis** hinges on 5 consequence/importance classes

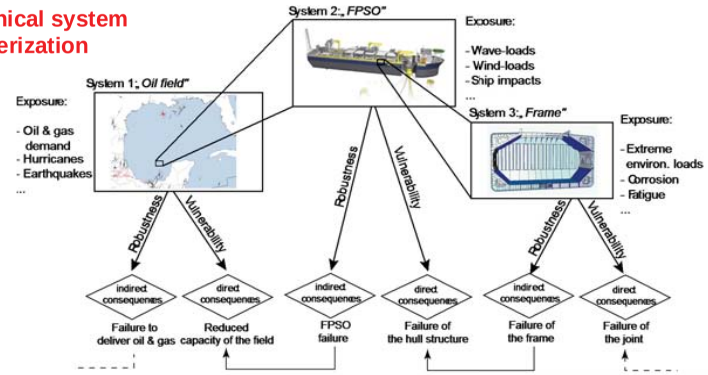
When failure consequences are well understood, then a **reliability-based assessment** can be used instead:

- target reliabilities for specific limit states and specified safety classes

A further simplification is permitted for design when failure modes and uncertain variables are well understood and can themselves be categorized: **semi-probabilistic design**

Systems and scenarios

Hierarchical system characterization



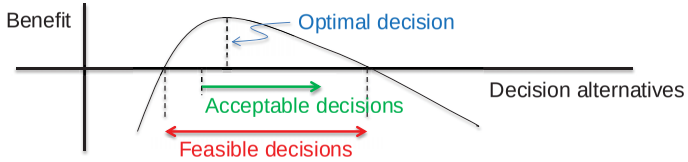
Scenario-based risk aggregation

actions events exposures → elements constituents → system process

direct consequences

indirect lifecycle consequences

Risk-informed constrained optimization as per ISO 2394:2015



"Feasible" i.e. yielding positive expected net (present value) utility

"Acceptable": acceptance criteria related to life safety and environment act as constraints

Important to note:

- additional robustness assessment is required for high consequence scenarios
- it assumes risk analyses to be performed in compliance with best practice (ISO standards on QRA, JCSS risk documents, annexes ISO 2394:2015)
- "benefits" depend on economics, incentives, financing and contractual conditions
- (life-cycle) costs include: design, QA/QC, construction, inspection, maintenance, down-time, repair, failure, compensation, insurance, business losses, decommissioning

2016 Offshore Structural Reliability Conference

Life safety: risk acceptance as per ISO 2394:2015

Life safety acts as a constraint on the risk-informed optimization

The basis for acceptance is the marginal life saving costs (MLSC) principle:

1. demonstrate that life safety risks are reduced by either technical, operational, and/or procedural measures to a level at which the cost of any further reduction would exceed limiting life saving costs
2. these limiting life saving costs are specified on the basis of socio-economic preference modeling ("revealed preference") and societal capacity for investment into life saving

Note that the MLSC procedure amounts to a quantifiable and operational version of the ALARP principle for life safety regulation

The MLSC also provides "lower-level" support for:

- the development of target failure probabilities in R-BD&A
- the calibration of design criteria in S-PD

Warning: MLSC may in some cases conflict with certain local risk acceptance regulations (typically Farmer-type criteria differentiating between individual and collective risk)

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Extreme and extraordinary hazards

A risk-informed approach makes it much easier and more transparent to communicate about extraordinary events such as Perfect Storms, Black Swans, Nightmare Scenarios, Unknowable Unknowns

Risk analysis (and decision making) is concerned with "what we know" and "what we need to know". Focusing on "what we do not know" runs counter to this methodology

Surprises and freak events are very low-probability high-consequence events which (beyond their explainable statistic of infrequency) can only be considered "freakish" with respect to a given state of knowledge and experience base

Events with very low probabilities ("extreme extremes" or "beyond extremes") do not form a limitation of rational decision making or risk analysis, neither in an aleatory nor in an epistemic sense

"Apocalyptic" events with catchy metaphoric names may mobilize the imagination but the real danger is that:

- they become a readily available excuse for incomplete decision making or poorly informed risk management
- they come to be viewed as actual obstacles or technical limitations in design and assessment
- they promote worst-case thinking, where:

Thinking	→	Imagining
Reason	→	Fear
Decision analysis	→	Speculation
Low-probability	→	Certainty (lack of probability)

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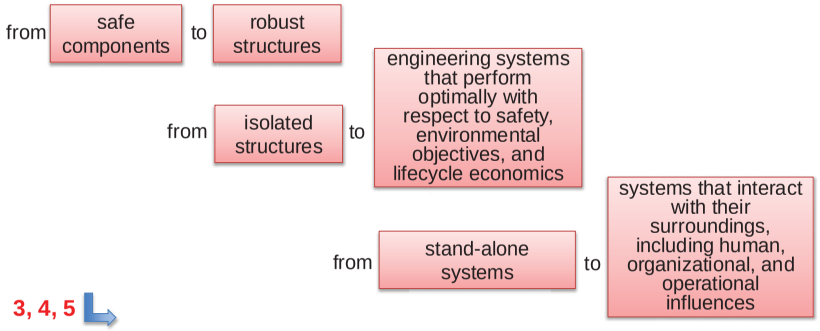
Five key benefits of risk-based standardization (1/2)

1. The holistic risk-informed approach integrates structural performance objectives, cost efficiency, and risk management in a supple and efficient way
2. The scope/scale of existing component-based design and assessment standards is widened and enriched:



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Five key benefits of risk-based standardization (2/2)

1, 2

3. The range of sustainable development objectives can be expanded
   - life safety
   - reliability, serviceability
   - quality of environment
   - cost efficiency given limited resources
   - effective use of energy, CO_2, natural resources
4. The risk-informed approach allows for consistent use of important tools such as:
   - pre-posterior scenario analysis
   - event precursor analysis (important for "perfect storms")
   - advanced Bayesian belief network analysis (for complex systems and mitigation strategies)
   - inclusion of human and operational error
5. It facilitates risk communication particularly with respect to extreme and/or exceptional events

6.2 Presentation by Farrokh Nadim

NTNU Norwegian University of Science and Technology

International Association of Oil & Gas Producers

Uncertainty Assessment of Geotechnical Design and Calibration of Resistance Factors for Offshore Piles

Farrokh Nadim
Norwegian Geotechnical Institute

The 3rd Offshore Structural Reliability Conference
OSRC2016
14-16 September, Stavanger, Norway

Motivation for the NGI JIP described in this presentation

- Offshore design guidelines (API, ISO) require the same level of safety for the new pile capacity methods (read CPT-methods) as for the API method.
- The designer is required to select an "appropriate" safety factor when using the CPT-methods.
- One can choose to be conservative and apply a "high" safety factor, or one can document the safety level and "calibrate" the required partial safety factors such that the target level of safety is achieved.
- Guidelines are needed for the definition and selection of characteristic soil strength parameters for design.

Axial pile capacity calculations methods considered in the study

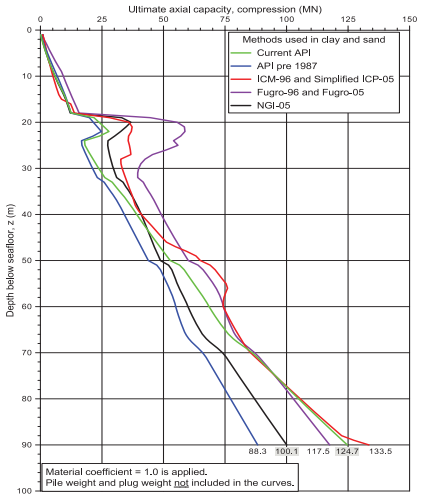
- Current API method

+ 4 CPT-based methods:

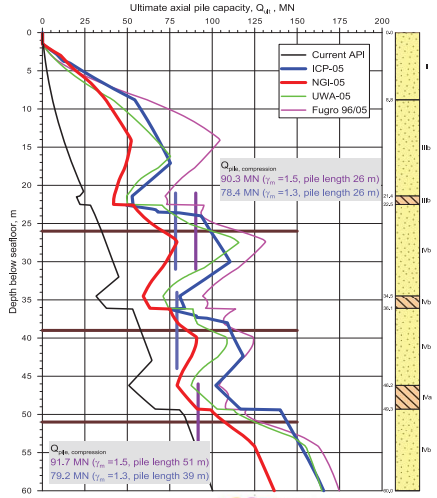
- NGI-05
- ICP-05
- Fugro-96/05
- UWA-05/13

Deterministic Q_{ult} with characteristic shear strength parameters

Predominantly Clay site



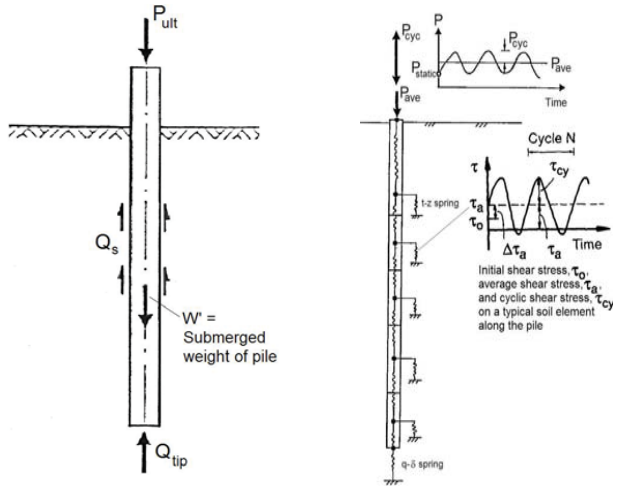
Predominantly Sand site



Calculation steps in case studies (1/2)

1. For each axial pile capacity calculation method considered, gather information about the soil parameters required for that method.
2. Do statistical evaluation of the required parameters (from direct measurements, derived form other parameters or obtained form literature, i.e. correlations) and do an estimate of the uncertainties.
3. Assess other factors affecting axial pile capacity, including the effect of cyclic loading and time after installation.
4. Establish the statistical model for the soil parameters of interest, including mean, standard deviation and probability density function.
5. Do a statistical analysis of the uncertainty in axial pile capacity associated with each design method used.

Typical axial loading of an offshore pile



Calculation steps in case studies (2/2)

6. Derive the probability distribution function of the axial pile capacity for each design method, including the uncertainties (inherent variability, measurement error, statistical uncertainties, etc.) in soil parameters, calculation model, effects of cycling loading on the soil resistance and the effect of time after installation (if relevant).
7. Assess the annual probability of failure for each calculation method, considering the probabilistic distribution of the loads and of the pile resistance.
8. Calculation of the required resistance factor for a target annual probability of failure (e.g. 10^{-4}/yr or 10^{-5}/yr).

Format of load and resistance factors

Design Criterion: $[\gamma_{l\,stat} \bullet P_{stat} + \gamma_{l\,env} \bullet P_{env}^{char}] \leq Q_{ult}^{char} / \gamma_m$

$\gamma_{l\,stat}$ = Load factor on static load

P_{stat} = Selected characteristic static load

$\gamma_{l\,env}$ = Load factor on environmental load

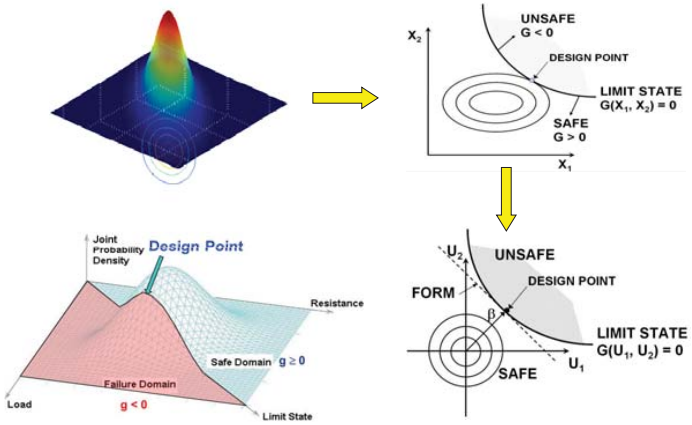
P_{env}^{char} = Selected characteristic environmental load (typically the 100-yr load, $P_{env}^{100\text{-}yr}$)

Q_{ult}^{char} = Characteristic ultimate axial pile capacity

γ_m = Resistance factor on characteristic pile capacity

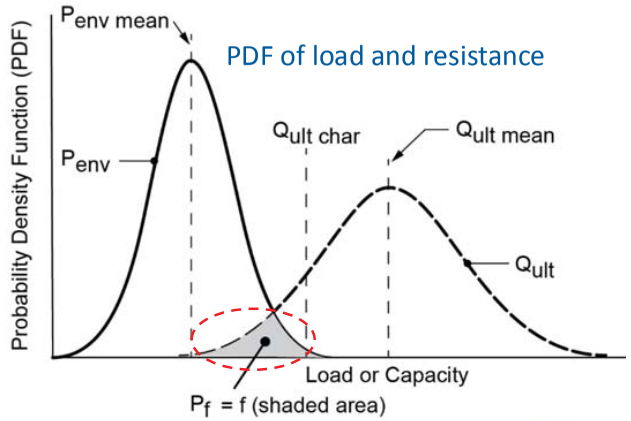
The JIP focuses on assessing the resistance factor γ_m in deterministic analyses that would ensure adequate safety with the CPT design methods for axial pile capacity.

FORM/SORM approach was used



Calibration of partial safety factors

$$Q_{ult}/\gamma_m \geq [\gamma_{l\,stat} \cdot P_{stat} + \gamma_{l\,env} \cdot P_{env}^{char}]$$



Probabilistic analysis with FORM / SORM

FORM / SORM analyses provide:

- Probability of failure, P_f
- Reliability index, β
- The most probable combination of parameters leading to failure (design point coordinates)
- Partial safety factor (obtained by dividing the characteristic value of a parameter by its design point coordinate)
- Sensitivity of P_f to any change in the random variables (sensitivity factors)

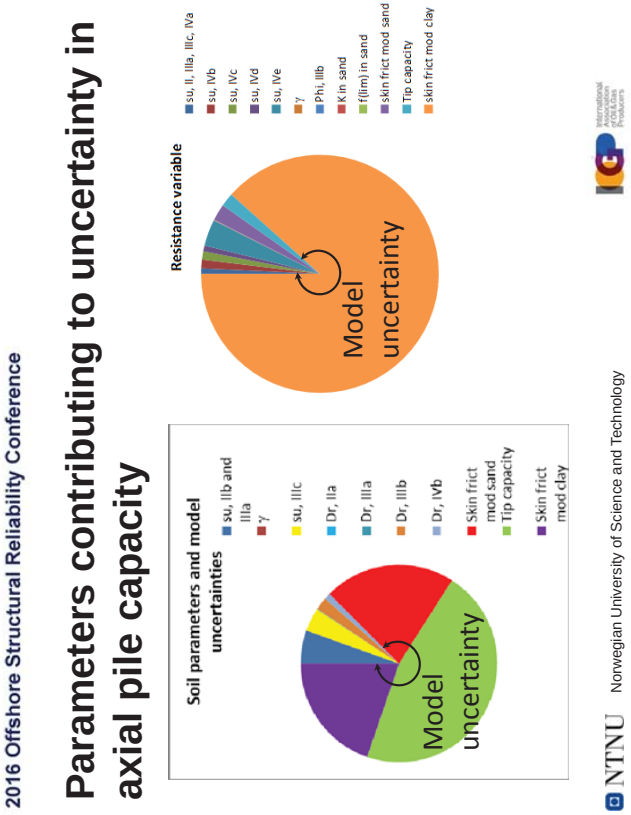
Consequence for required pile penetration depths when analysed from a reliability-based design perspective

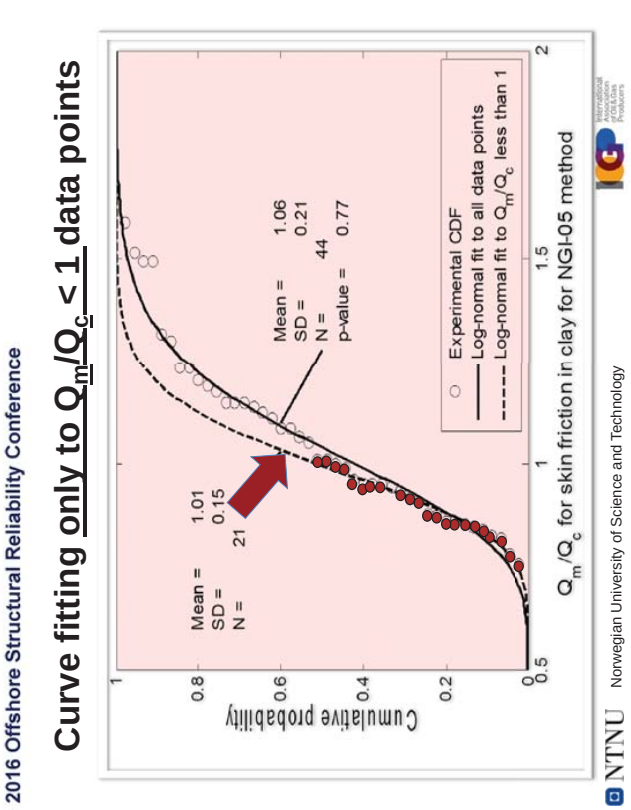
Method	Required pile penetration depths		
	Site A (clay)	Site B (sand)	Site C (clay and sand)
NGI-05	90m to 75m*	51m to 27m*	45m to 36m*

* Reduction in penetration depth with design for target $P_f \leq 10^{-4}$/yr (NGI-05 CPT method used)

after Lacasse et al (2013)

Potentially huge savings, several millions US$ at each site!


2016 Offshore Structural Reliability Conference
Parameters contributing to uncertainty in axial pile capacity
Soil parameters and model uncertainties
su, IIb and IIIa
γ
su, IIIc
Dr, IIa
Dr, IIIa
Dr, IIIb
Dr, IVb
Skin frict mod sand
Tip capacity
Skin frict mod clay
Model uncertainty
Resistance variable
su, II, IIIa, IIIc, IVa
su, IVb
su, IVc
su, IVd
su, IVe
γ
Phi, IIIb
K in sand
f(lim) in sand
skin frict mod sand
Tip capacity
skin frict mod clay
Model uncertainty
NTNU
Norwegian University of Science and Technology

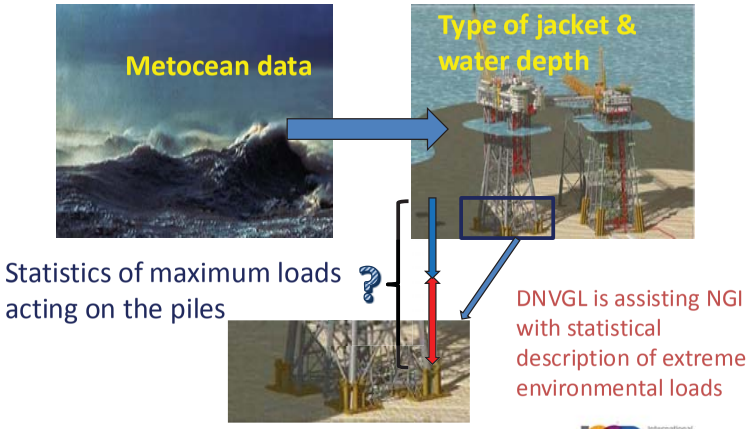

2016 Offshore Structural Reliability Conference
Curve fitting only to Q_m/Q_c < 1 data points
Mean = 1.01
SD = 0.15
N = 21
Mean = 1.06
SD = 0.21
N = 44
p-value = 0.77
Experimental CDF
Log-normal fit to all data points
Log-normal fit to Q_m/Q_c less than 1
Cumulative probability
Q_m/Q_c for skin friction in clay for NGI-05 method
NTNU
Norwegian University of Science and Technology

2016 Offshore Structural Reliability Conference
Challenge: Quantification of model uncertainty
Mean = 1.01
SD = 0.15
N = 21
Mean = 1.06
SD = 0.21
N = 44
p-value = 0.77
Experimental CDF
Log-normal fit to all data points
Log-normal fit to Q_m/Q_c less than 1
Cumulative probability
Q_m/Q_c for skin friction in clay for NGI-05 method
NTNU
Norwegian University of Science and Technology

2016 Offshore Structural Reliability Conference
Statistical description of environmental loads acting on the pile (case study with comprehensive load data)
0.1/0.01
0.1/0.0001
0.01/0.001
0.01/0.0001
0.001/0.0001
-ln(-ln(Fx(x)))
Extreme value
Possible Gumbel distributions for annual maximum environmental load (CoV = 10-15%)
NTNU
Norwegian University of Science and Technology

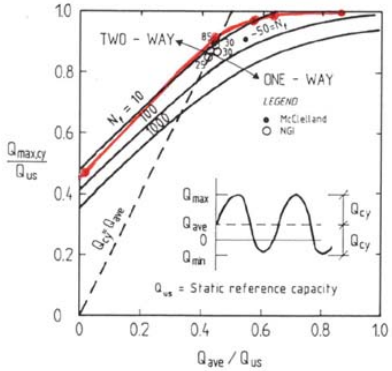
Challenge: Statistical description of loads

Can we generalize and develop reasonable distribution functions for extreme loads when data are lacking?



Challenge: Assessment of the effects of cyclic loading on axial pile capacity

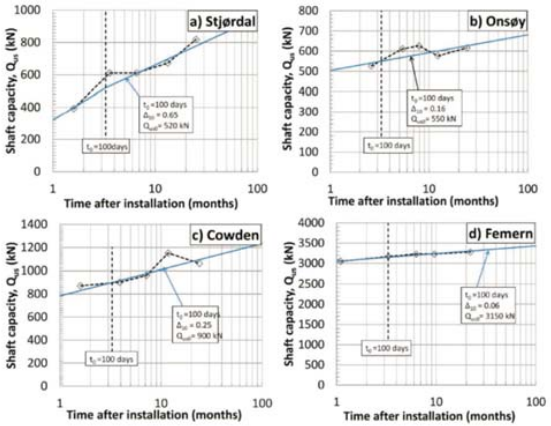
NOTE: Cyclic capacity is not a unique value. It depends on many factors like soil response to cyclic loading, drainage conditions, loading history, ratio of permanent to cyclic loads, rate of loading, load path to failure….



Challenge: Assessment of change in axial pile capacity with time after installation

Examples of ageing effects on ultimate skin friction in clay (Karlsrud et al., 2014)

Recent research shows that there are also significant time effects in sand



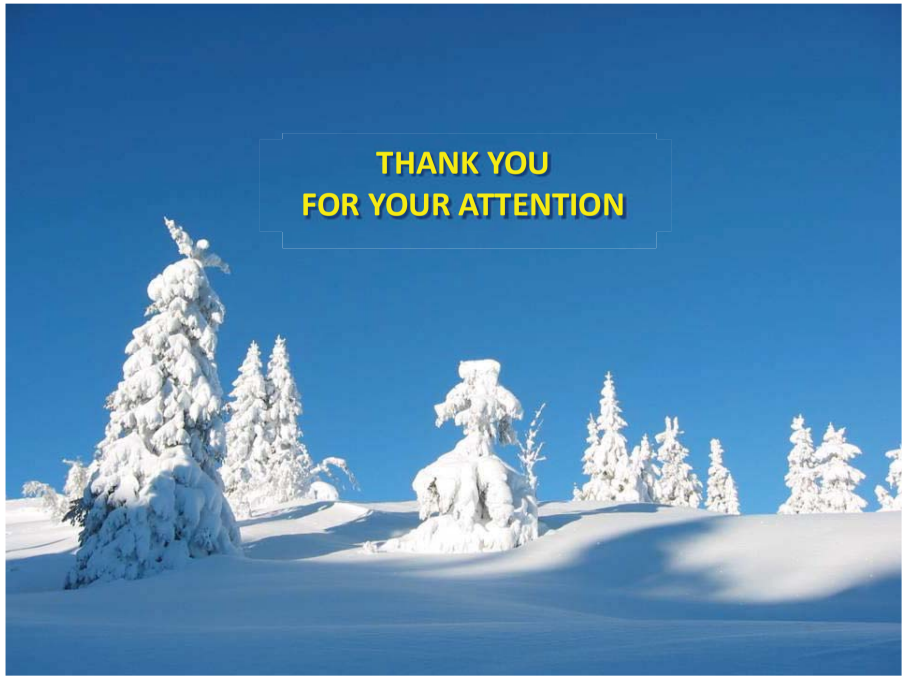
JIP sponsors and timeline

- Project start: May 2014
- Expected completion date: December 2017
- Sponsors:

  Statoil, Lundin, Dong Energy, ONGC, Petrobras, DNVGL (case study data are provided by sponsors).
- **New sponsors are welcome**

Status of JIP technical activities

- Unified database of pile load tests
  - Database will be ready in October 2016.
  - Re-analysis of model uncertainties to be completed by Dec. '16 for piles in sands and clays
- Case studies
  - 9 case studies completed and 2 are underway.
  - All case studies to be updated when model uncertainties are re-evaluated (Oct. '16 - March '17).
- Recommendations
  - Key activity now is dissemination of information regarding the JIP.
  - Recommendations to be proposed for standards and guidelines at the end of the JIP in Dec. 2017.



Chapter 7

Session 5: Accidental Collapse Limit State

7.1 Presentation by Dave Wisch

NTNU
Norwegian University of Science and Technology

International Association of Oil & Gas Producers

Standards – Targets and not Risk Management

Dave Wisch
Chevron Energy Technology Company

The 3rd Offshore Structural Reliability Conference
OSRC2016
14-16 September, Stavanger, Norway

Topics

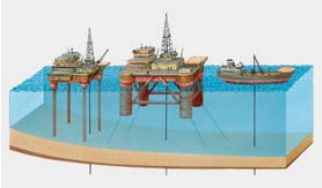
- "Way Back When"
- Setting the Stage
- Present Day
- Incidents & One Perspective
- Focus on Analysis
- Role of Standards
- Where Do We Go

Setting the Stage

- Examples are Structural and Mechanics
- Items selected for concept, many equivalent
- Spans all disciplines
- Relying more on machines than people?
- Do we need to adjust focus?

Setting the Stage

- Examples are Structural and Mechanics
- Items selected for concept, many equivalent
- Spans all disciplines
- Relying more on machines than people?
- Do we need to adjust focus?
- Performance not matching computed expectations



Way Back When

- Focus on Design
- Much Empirical
  - Pilehead Damage Table'
  - D/t Ratios
  - "K" factors
  - Etc
- Tape Measure Tolerances
- Constructability
- Many Unanalyzed Conditions – "Rules of Thumb" Worked
- Significant Life Cycle Input
- Slide Rule Accuracy "Good Enough"
- Good Understanding of Context

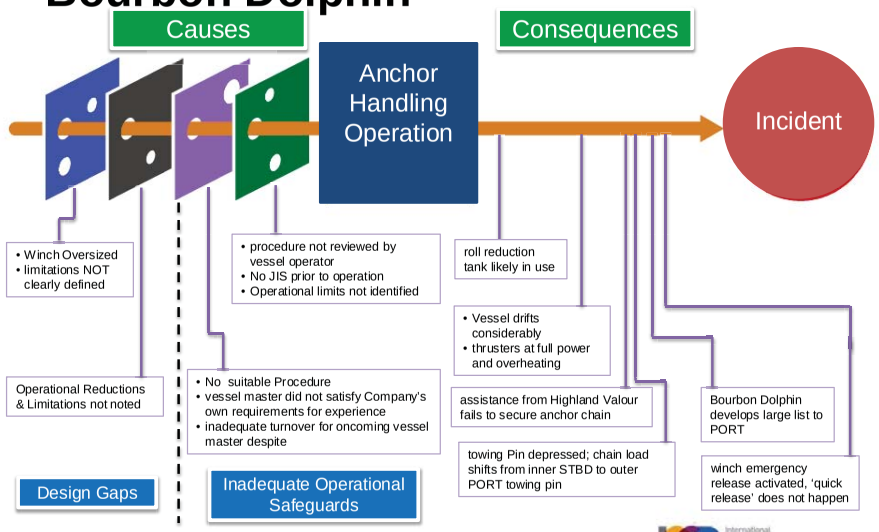
Present Day

- Sophisticated Software
- Heavily Reliant on Analysis
- Increasing Use of Analysis as Design
- Increasing Segmentation and Experts
- Direct Computation Replacing Empirical
- Less Understanding of Software Limitations
- Increasing Acceptance of Input from Other Specialists
- "Optimization" Becoming Standard Practice
- Less Life Cycle Experience

Incidents

- Ocean Ranger
- Alexander Kielland
- Sleipner
- Bourbon Dolphin
- Typhoon
- Joliet Tendon Loss
- P-36
- Mighty Servant – Nemba
- White Rose
- COSC Innovator
- MODU's – Ivan/Katrina/Rita
- Jackets – Andrew
- Jackets – Ivan/Katrina/Rita

Bourbon Dolphin



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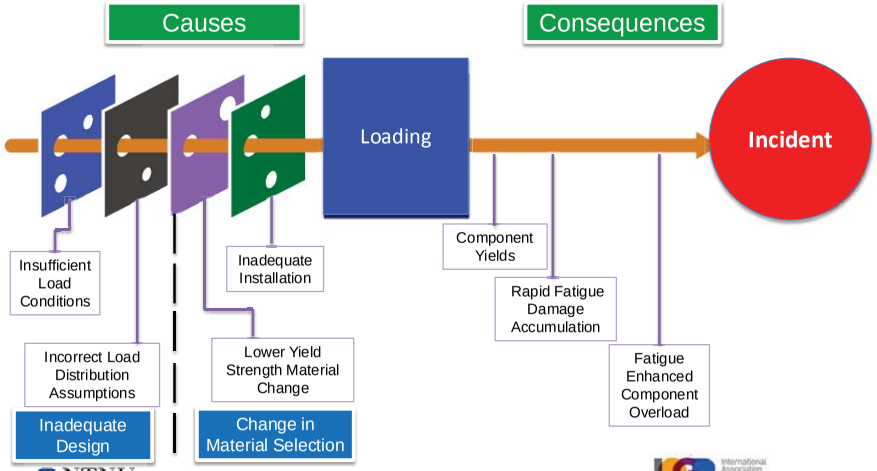
Case 1



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Design Gaps

- "Met Specs/Standards"
- Bourbon Dolphin
  - Oversized Equipment Relative to Operating Stability
  - High Capacity within Limits, little to no margin outside limits
- Case 1
  - Inadequate Design Conditions
  - Incorrect Assumptions in Loadings
- COSL Innovator
  - Insufficient Air Gap
  - Insufficient Consideration of Local Wave Pressure/Loads
  - Insufficient Design/Detailing of Penetration

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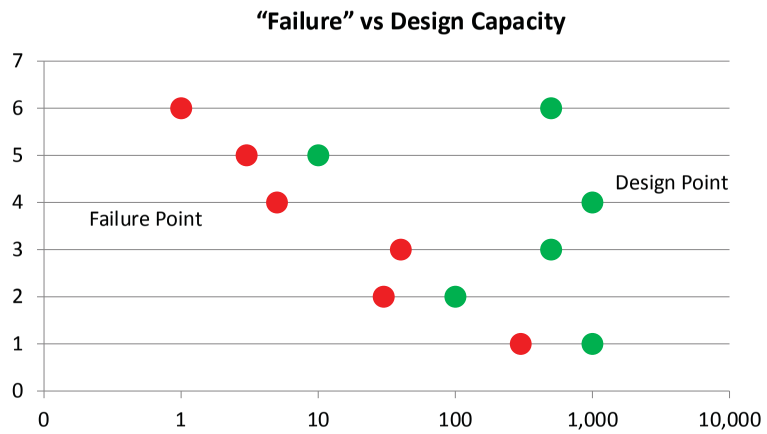
Plot of Failure Vs. Target



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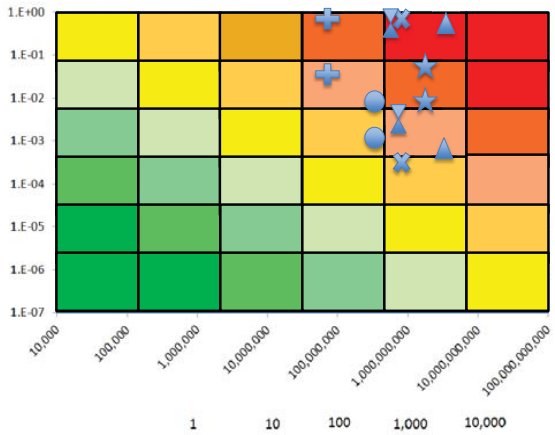
Risk Management

Pre-Service & Operations, $ & Life Safety



Incidents

Unexpected

- Ocean Ranger
- Alexander Kielland
- Sleipner
- Bourbon Dolphin
- Typhoon
- Joliet Tendon Loss
- P-36
- Mighty Servant – Nemba
- White Rose
- COSC Innovator

Expected

- MODU's – Ivan/Katrina/Rita
- Jackets – Andrew
- Jackets – Ivan/Katrina/Rita

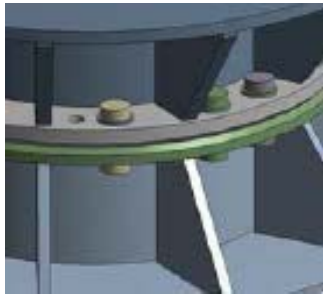
Common Elements

- High Consequence
  - Loss of life
  - Headline
  - Impact on Industry
- Incident below Predicted
- Analysis Missed "Failures"
  - Analysis Correct in Most Cases
  - Missed Loads or Faulty Interpretation
  - Scenario Outside of Software Capability
- Design Issues
  - Little to no context of loads, load variability, sensitivities
  - Lack of understanding of Constructability
    - Lack detail, impossible tolerances and/or criteria, etc.

Context and Understanding

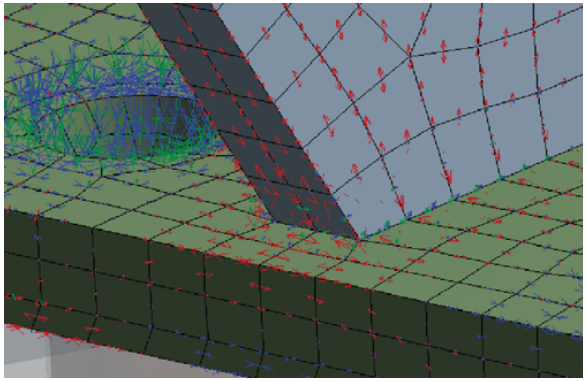
Fatigue Design – Allowable is a "Given Value"

As Built

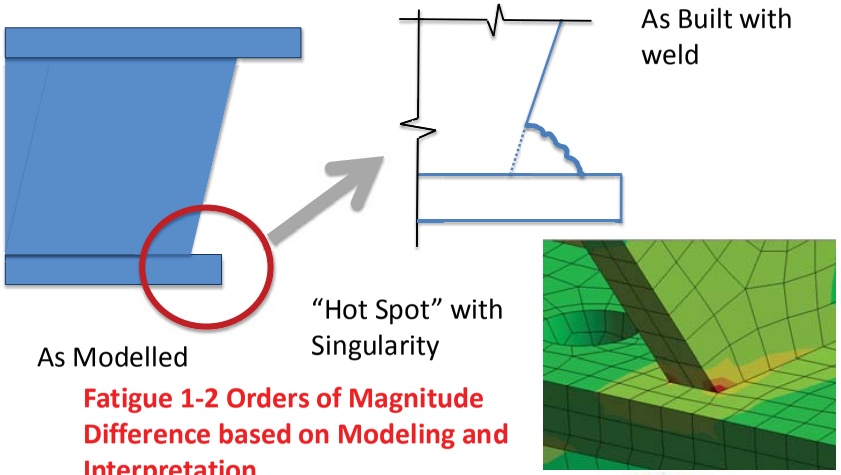


Graphic

Great Analytic Capability



Context and Understanding



Fatigue 1-2 Orders of Magnitude Difference based on Modeling and Interpretation

Standards - Morphing

- 1970's –
  - Experience
  - Empirical
  - Emerging Analytical Tools
  - Design & Performance Centric
- 2010's –
  - Analytical Focused
  - "Targets" more emphasized than performance
  - Better understanding of environment
  - Desire for more and more exactness
  - Increasing sub-optimization

Some Food for Thought

- Practice
  - Analytic capability impressive
  - Complexity in day to day practice increasing
  - Many specialists and subspecialists
  - Replacing "rules of thumb" with direct computation
- Performance
  - Many significant incidents in conditions well below "Expected"
  - Missed design conditions
  - Impractical tolerances and criteria leading to:
    - Increased cost
    - Changes impact performance but not cycled through designer
- **Industry Performance not Meeting Reliability "targets"**
- **Managing Numbers/Targets --- Not Risk**

Where to Go

- Increase practical design guidance in standards
- Develop framework in lieu of detailed recipes in select areas
- Focus less on defining and computing very rare event conditions
- Adjust standards to eliminate early "failures"
- Adjust mindset with refocus on design
  - Focus on analytical targets leading to misses

7.2 Presentation & article by Gerhard Ersdal, Torgeir Moan & Jørgen Amdahl

NTNU Norwegian University of Science and Technology

International Association of Oil & Gas Producers

Assessment of ship impact risk to offshore structures New NORSOK N-003 guidance

Torgeir Moan, Jørgen Amdahl, Gerhard Ersdal

The 3rd Offshore Structural Reliability Conference
OSRC2016
14-16 September, Stavanger, Norway

Outline

- Introduction
  - ALS design check
- Experience with ship collisions and the 10^{-4} event
  - Vessel size
  - Vessel velocity
- Recommended impact energy associated with attendant vessels
- Structural damage assessment of ship collisions
  - ALS check according to NORSOK N-001
  - Vessel configuration
  - Force-deformation curves
- How large collision can a typical NS structure resist?
- Is strength design possible?
- Examples of analysis of impact on jacket, semi, and jack up
- Conclusions

Introduction

ALS design check for ship impacts requires consideration of 10^{-4} and 10^{-2} events

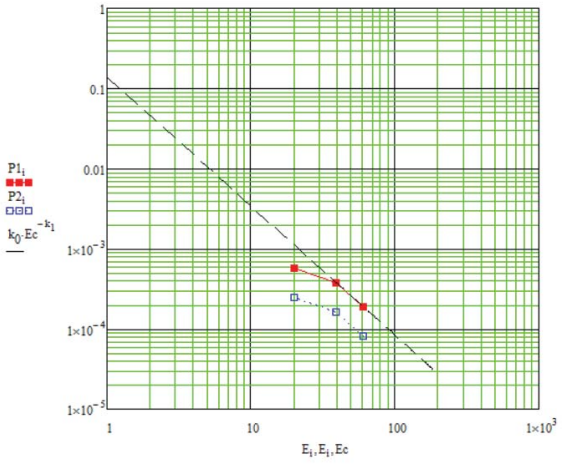
After the impact

Ship collision and impact energy – attending vessels

Experience with ship collisions

50 MJ

The 10^{-4} event related to attendant vessels



Assumption 1:
N=350*15

Assumption 2:
N=350*35

Number of platforms on NCS and UKCS: 350

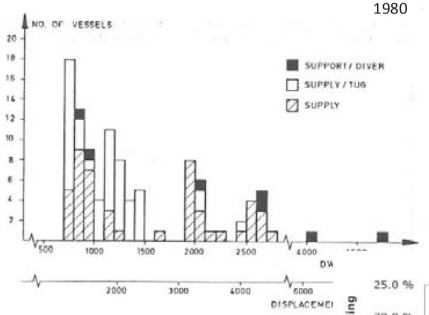
North Sea UKCS and NCS collisions in the period 2000-2016

- West Venture 2004 → 39 MJ
- Ekofisk 2/4-P 2005 → >20 MJ
- Grane 2007 → Low
- Ekofisk 2/4-W 2009 → 70 MJ

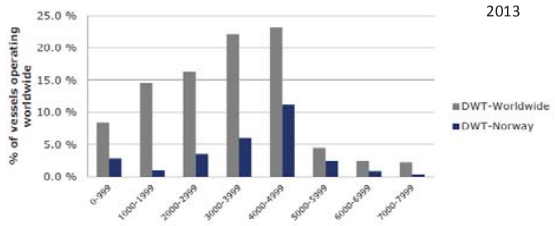
A. Kvitrud OMAE

Comparison of attending vessel size

1980



2013



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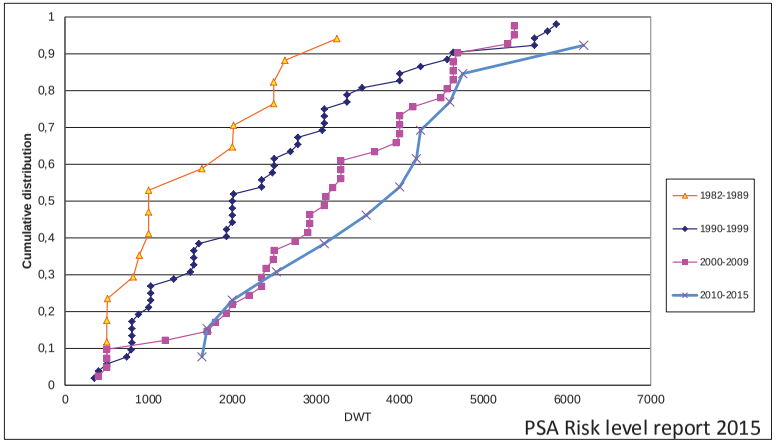
Development of attending vessel size



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Vessel velocity – head on collsion

v=3 m/s

Impact energy: $E=1/2 * m*v^2 = ½ * 10\ 000\ 000 * 1.1 * 9 = 50$ MJ

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Vessel velocity

v=? m/s

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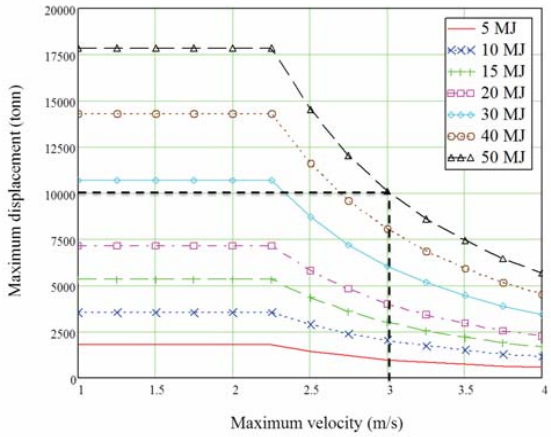
Recommended impact energy in head-on impact in NORSOK N-003



Structural damage assessment of ship collisions

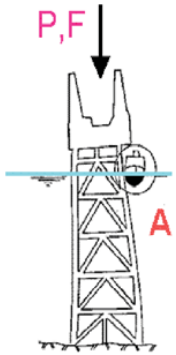
Not only size and speed matter

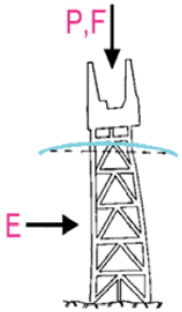
OSV bow configurations

Before:
Raked stem or small bulb

Now:
Bows of different shapes and bulbous
Ice strengthened

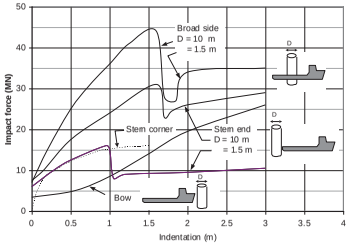
Accidental Limit State check according to NORSOK N-001





- Initial damage caused by ship impact
- Residual strength of damaged structure wrt to environmental loads

Force-deformation curves for supply vessels - NORSOK N-004 App A/ DNVGL RP-C204

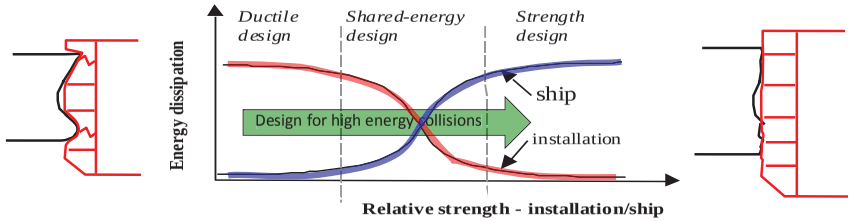


- Force curves developed for **raked bow** in 1981 - not representative for large vessels, different bow shapes and ice strengthened ships
- Significant work on relevant ships conducted in recent years based on NLFEA
- Revised DNVGL-RP-C204 with new force curves to appear soon

How large collisions can typical offshore structures resist?

How can we design against collisions?

Energy dissipation vs. relative strength for ship bow/FPSO side

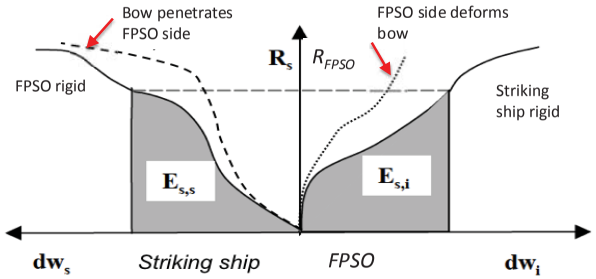


- Ductile design - Bow penetrates FPSO side
- Shared energy design - Bow and FPSO deform
- Strength design - FPSO crushes bow

Fairly moderate modification of relative strength may change the design from ductile to strength or vice verse

Interaction effects on strain energy dissipation

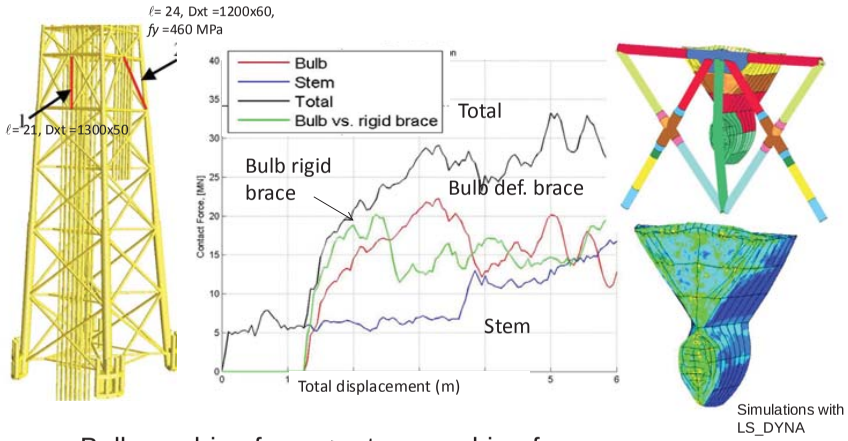
Revised DNVGL RP-C204



- Dissipated energy equals area under force-deformation curve
- If the impacting object is assumed to be deformable, the interactive nature of the resistance (R) of the two structures should be recognised

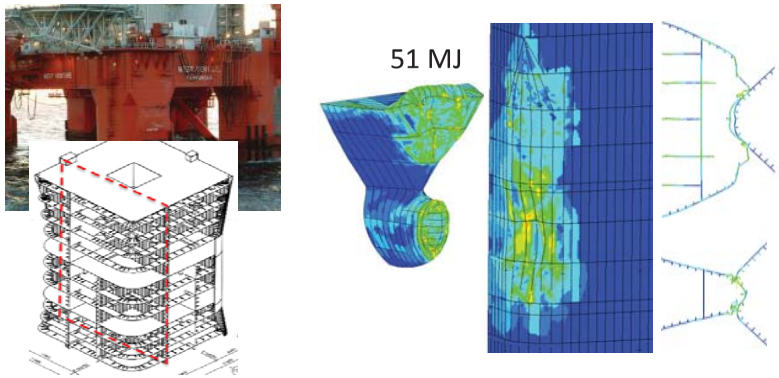
Is strength design possible?

Bow collision with jacket braces



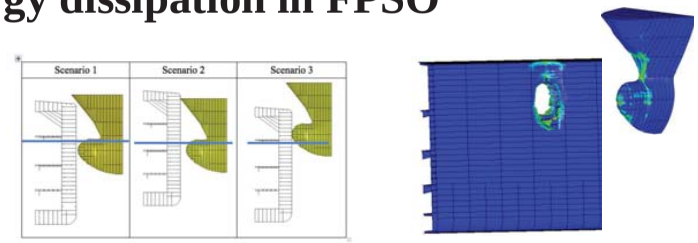
- Bulb crushing force > stem crushing force
- Even bulb crushed with moderate deformation of brace

Bow collision with semi-submersible column



- Forecastle relatively weak- small deformations of column (ref West Venture 2004)
- Bulb crushes partly, but penetrates column – fracture predicted for 37 MJ
- Forecastle deck "overhang" of importance
- Flooding of tanks to be considered

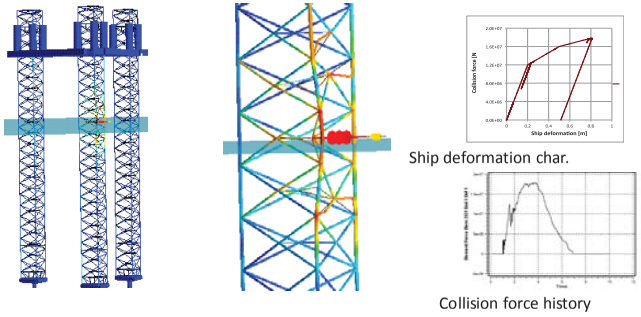
Energy dissipation in FPSO



Cargo tank hole 32 MJ > 90 MJ 22 MJ

- Bow superstructure crushed – but above deck for FPSO fully laden (Scenario 3)
- Bulb penetrates FPSO side
- Penetration of cargo tank in Scenario 3 for 22 MJ
- Bulb protrusion can have a big impact on damage

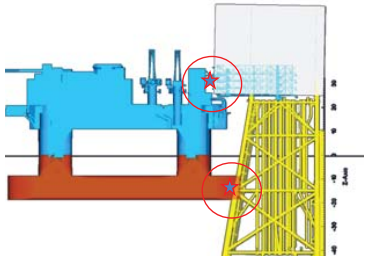
Energy dissipation in jack-ups



- North Sea jackups are compliant - eigenperiods ~ 5-8 s
- Dynamic analysis required
- Energy dissipation in ship 22 MJ, local plastic 25 MJ, global elastic 20 MJ (!)
- The jack up seems capable of surviving high energy impact, but joint -, strain details were not checked

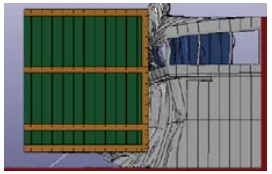
"New" collision scenarios

not covered by NORSOK N-004 App. A



Collision between jacket and floatel after DP failure

Accommodation unit to be considered?



290 MJ

Collision between wheelhouse and platform deck

Conclusions

- Collision risk controlled by reducing probability and reducing consequences by design
- Minimum energy by attendant vessel impact is recommended
- For jacket braces strength design may be needed
- Jack-up compliance favorable wrt energy absorption
- Semi-sub column and FPSO side may crush forecastle, but bulb may penetrate – (strength design is possible)
- Penetration of cargo tanks/flooding of void spaces important issues
- What compartmentalization is needed?
- OSV size, bow shape and any ice-strengthening must be taken into account – new collisions scenarios
- Revised design rules/RP needed (and is in the pipeline)

Assessment of ship impact risk to offshore structures - New NORSOK N-003 guidelines

Torgeir Moan[a], Jørgen Amdahl[b], Gerhard Ersdal[c]

[a] Centre for Autonomous Marine Operations and Systems, Centre for Ships and Ocean Structures, Norwegian University of Science and Technology (NTNU), Trondheim, Norway

[b] Centre for Autonomous Marine Operations and Systems, Norwegian University of Science and Technology (NTNU), Trondheim, Norway

[c] Petroleum Safety Authority, Norway

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Assessment of ship impact risk to offshore structures - new NORSOK N-003 guidelines

Torgeir Moan[1], Jørgen Amdahl[2], Gerhard Ersdal[3]
[1]Centre for Autonomous Marine Operations and Systems, Centre for Ships and Ocean Structures, Norwegian University of Science and Technology (NTNU), Trondheim, Norway
[2]Centre for Autonomous Marine Operations and Systems, Norwegian University of Science and Technology (NTNU), Trondheim, Norway
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ABSTRACT: The regulatory requirements for the Norwegian Continental Shelf specify that ship impact actions and other accidental actions should be determined by risk assessment. However, when the first requirements on collision energy from supply vessels were introduced by DNV around 1980, the frequency of impacts by attendant vessels were high – of the order of 10^{-3} per installation year. Therefore, it was assumed in the initial requirements that the impact action associated with attendant vessels should, as a minimum, be calculated for the maximum authorized vessel assumed to service the installation. At that time the resulting minimum impact energies were 11 and 14 MJ for head on and side impact, respectively; and have remained the same since then. However, the supply vessel size has since increased and design of supply ship bow and platform has changed. Further, the use of DP controlled supply vessels has increased, which may imply larger velocities at impact. Moreover, the consequence of ship impacts might change e.g. due to the change in design of supply vessels by providing ice-strengthened bows in supply vessels and platforms with cantilevered decks. In the revision of the NORSOK N-003 standard on "Actions and action effects" the requirements to ship impacts are being reassessed and updated based on statistics on supply vessel sizes and collision energies; as well on service experiences. Besides revisiting the requirements to attendant vessels, other ship impact scenarios are also considered. This especially includes the collision risk associated with shuttle tanker – FPSO. This paper presents the background for the revised standard; in terms of ship impact actions relating especially to supply vessels and shuttle tankers, recognising that the main risk control relating to ship impact is to limit the probability of impacts by operational control. Moreover, the consequences in terms of damage for different types of platforms are addressed, by e.g. demonstrating the feasibility of satisfying more restrictive requirements and especially the effect of providing ice-strengthened bows in supply vessels and designing platforms with cantilevered decks.

KEY WORDS: Ship impacts, risk, ALS, NORSOK standard

1 INTRODUCTION

According to the regulations NORSOK Z-013 and NORSOK S-001for the Norwegian Continental Shelf accidental actions on platforms should be assessed by a risk assessment - with due account of the factors of influence. Such factors may be personnel qualifications, operational procedures, the arrangement of the installation, equipment, safety systems and control procedures. In the design phase particular attention should be given to layout and arrangement of the structure and equipment in order to minimise the adverse effects of accidental events; e.g. by ensuring that risers and conductors are not subjected to ship impact damage.. To limit the risk of total loss they should be applied in the ALS design check - and refer to events with an annual exceedance probability of 10^{-4} [1].

When the first ship collision considerations were introduced by DNV around 1980, the frequency of impacts by attendant vessels were high – of the order of 10^{-2} per installation year. In the late 90s Haugen [2] estimated the annual exceedance probability corresponding to the minimum impact head-on event to be about 10^{-3} for visiting vessels in the North Sea. Therefore, it was assumed in the initial requirements that the impact action associated with attendant vessels should, as a minimum, be calculated for the maximum authorized vessel assumed to service the installation. At that time the resulting minimum impact energies were 11 and 14 MJ, respectively, for head on and side impact; and remained the same since then. However, the supply vessel size has increased and the design of supply ship bow and platform has changed. Moreover, the use of DP controlled supply vessels has increased, which may imply larger velocities at impact. Moreover, the consequence of ship impacts might change e.g. due to the change in design of supply vessels by providing ice - strengthened bows in supply vessels and platforms with cantilevered decks.

This paper deals with the assessment of impact actions and their effects by oil field related supply vessels and shuttle tankers (during off-loading). However, other potential ship impact scenarios exist; including merchant vessels, shuttle tankers and floating platforms.

2 SHIP IMPACT ACTIONS

2.1 General

Ship impact actions are characterized by kinetic energy, impact geometry and the relationship between action and indentation. Ship collision scenarios can be categorized into two groups for different types of ship operations as shown in Table 1, namely:

- Powered collisions (Vessel steaming towards the installation)

- Drifting collisions (Vessel drifting towards the installation)

Table 1 Categories of colliding vessels

	Traffic category	Vessel category	Remarks
External	Merchant	Merchant ships Cargo, ferries etc.	Commercial traffic passing the area
	Naval traffic	Surface vessel	Both war ships and submarines ines
		Submerged vessels	Submerged submarines
	Fishing vessels	Fishing vessels	Sub-categorized into vessels in transit and vessels operating in the area
	Pleasure	Pleasure vessels	Traffic passing the area
Field related	Offshore traffic -in the field -to/from other fields	Standby boats Supply vessels Working vessels	Dedicated standby boats Visiting supply vessels Special service vessels , e.g. for diving
		Offshore tankers	Shuttle tankers visiting the field
		Tow	Towing of drilling rigs, flotels, etc

Powered collisions involve scenarios caused by navigational/maneuvering errors (human/ technical failures), watch keeping failure, bad visibility/ineffective radar use, etc. A drifting vessel is a vessel which has lost its propulsion or has experienced a progressive failure of anchor lines or towline and is drifting only under the influence of environmental forces.

In view of the accident causation mechanisms it is reasonable to deal with two types of ship impact events; namely external – passing vessels and field related vessel impacts.

As mentioned above, the ship impact actions should in principle be determined by risk analysis. Risk-analysis for offshore facilities is outlined e.g. by Vinnem [3]. However, it should be noted that extensive experiences with accidental actions for typical platforms have led to specific actions [4].

The collision risk associated with route-based traffic is estimated based on two the following types of scenaria:

- Powered collisions. Vessel is steaming towards the installation without the crew being aware of the situation.
- Drifting collisions. Vessel is out of control and drifting towards the installation under the influence of environmental factors.

The collision risk associated with powered passing vessels can be estimated by various risk analysis models [2,5,6,7,8] The vessels are assumed to move in shipping lanes that are characterized by (1) number of vessels, (2) probability distribution across the lane, (3) size of the installation and (4) probability of failure of watch-keeping (for various reasons) for ships on a collision course. The model has been validated by comparison with field observations [6].

Various efforts to reduce the risk associated with ship impact should be considered and taken into account in the risk assessment. Such effort may be directed towards reducing the probability or the consequence of the impact. Reduction of the impact probability has been achieved by radar surveillance systems and by the provision of a 500 m radius designated safety zone around the platform with restrictions on entry by unauthorized vessels. Normally the operational control is assumed to be sufficient to handle the risk associated with passing vessels; implying that they do not lead to design accidental actions.

Due to the impact of a 550 t German submarine on the Oseberg B jacket in 1988, there has been some discussion about the potential hazards of submarine impact, e.g. on sensitive components like TLP tethers. However, the effect of both submarines and other naval vessels are normally neglected.

In this paper the focus will be on impacts by field-related vessels.

2.2 Attendant vessels

The initial ship impact requirements were based on statistics of the displacement of the classed supply ships in the DNV portfolio as shown in Figure 1. A maximum supply vessel size of around 3000 DWT (Dead Weight Tonnes), giving a maximum displacement of 5000 tons was then determined as representative. By accounting for added mass and an impact velocity of 2 m/s the minimum impact for design checks were established to be 11 and 14 MJ, respectively, for head on and side impact in the accidental limit state (ALS).

The present situation is rather different from that in 1980. Presently larger supply vessels are being used as illustrated in Figure 2. Whereas the majority of the supply vessel sizes in 1980 were in the range 1000 DWT to 2000 DWT, the majority of the supply vessels in 2013 were from 4000 to 5000 DWT. A maximum size of supply vessels based on Figure 2 is 8000 DWT in 2013 compared to ~3000 DWT in 1980.

This change of vessel mass fact implies an increase in the head-on impact energy from 11 to 29.4 MJ.

The ratio between displacement and DWT seems to have increased from about 1.65 to 1.75 [10]. The maximum ratio is as high as 2.4. Moreover, the assumed speed of the supply vessels may have changed as a result of the use of DP (dynamic positioning), increased engine power etc.

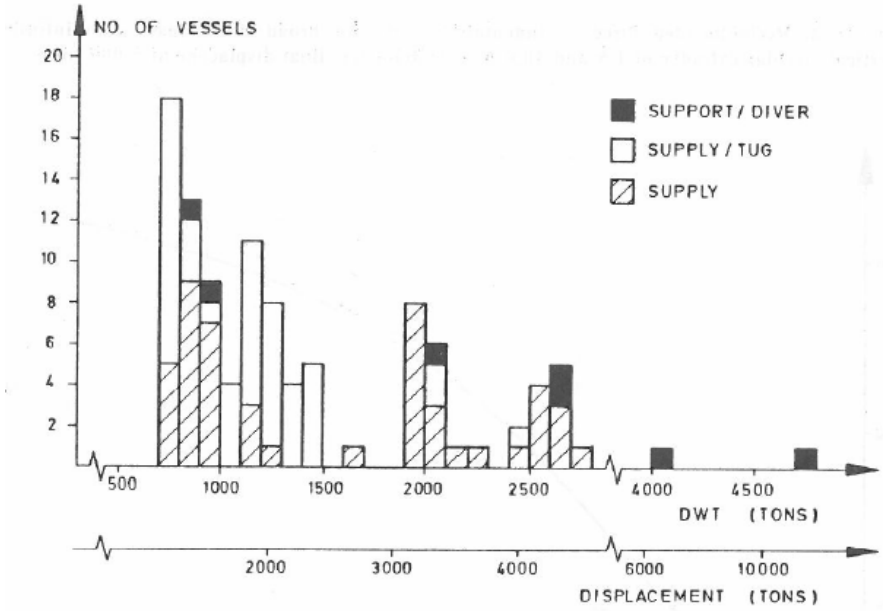

Figure 1: Supply vessels with DnV class, 1980 [9]

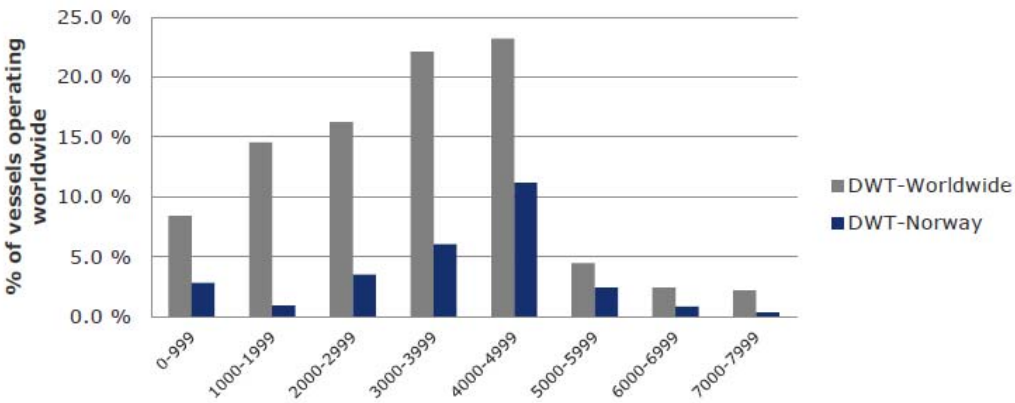

Figure 2: Overview of the DWT for 2013 [10]

J.P.Kenny [11] carried out the most comprehensive study of attendance vessel collisions, covering the period 1975-1986. HSE summarized collisions events in the period 1975 to 2001 on the UKCS [12]. The majority of the 557 incidents referred to attendant vessels, corresponding to annual frequency of 0.017. The frequency is high. However, most of these incidents were contacts with limited consequences. The main focus should be on the frequency of higher energy events. Table 2 shows accidents in the North Sea in the period 2000-2010.

Assuming these to be the most severe collisions in the UK and Norwegian offshore petroleum industry, and further assuming that there are about 5200 platform years on UKCS and NCS in the period 2000-2015, the probability of exceedance of 20, 39 and 60-70 MJ is $3/5200 \approx 6{\cdot}10^{-4}$, $4{\cdot}10^{-4}$, $2{\cdot}10^{-4}$, respectively. The value corresponding to a relative frequency of $1{\cdot}10^{-4}$ is then about 80 MJ. If these events are considered to be the largest events in the time of operations in Norway and UK, the relative frequency becomes 1/2 of the frequencies estimated above. The 10^{-4} event then corresponds to about 60-70 MJ.

With a characteristic value of dead weight of vessel ships ~8500 tonnes, a ratio between maximum displacement and DWT of 1.75, and added mass coefficient of 0.1 a velocity of 3 m/s, the updated data indicate a head-on supply vessel collision energy of 73.6 MJ, quite similar to the 70-80 MJ inferred above. However, it should be noted that 2 of the 3 collisions are on the Ekofisk field with many platforms and hence many visiting ships. As a result the value of 75 MJ may be slightly conservative for a single platform. The effect of this accumulated collision risk on fields with several or few

platforms are not sufficiently studied to give precise estimates yet. Hence, these estimates are clearly very crude.

Table 2. North Sea UKCS and NCS collisions after 2000 [13]

	Estimated speed (m/s)	Estimated displacement (tons)	Estimated collision energy (MJ)
Far Symphony vs West Venture March 7th 2004	3.7	5000	39
Ocean Carrier vs Ekofisk 2/4-P June 2nd 2005	3	4679 (DWT)	20
Bourbon Surf vs Grane July 18th 2007	1-3.5	3117 (DWT)	Low
Big Orange XVIII vs Ekofisk 2/4-W June 8th 2009	4.5-4.8	6000	60-70
Far Grimshader vs Songa Dee January 18th 2010	Low		Low

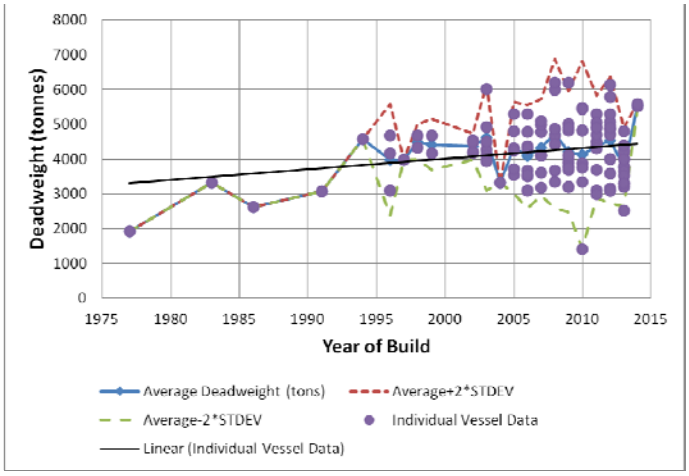

Figure 3. Vessels operating on the NCS [14]

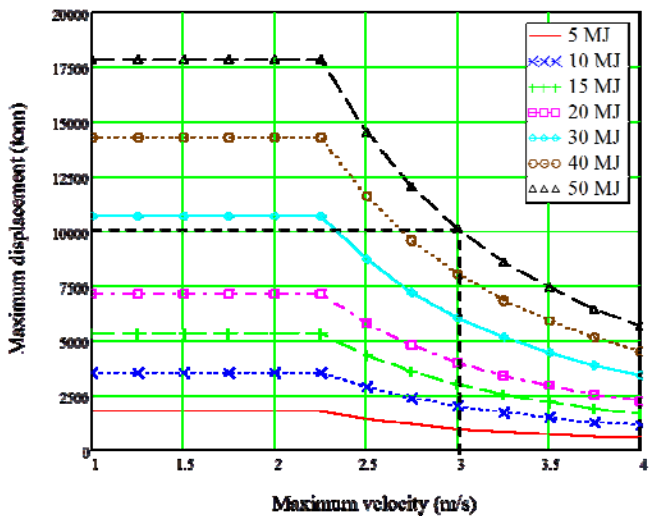

Figure 4: Operational restrictions in the safety zone - acceptable combination of vessel size and velocity is found beneath the respective curve for the documented impact energy capacity (MJ).

Unless operational restrictions on the vessel size and maximum speed are enforced, the minimum impact energy in bow impact is suggested to be 50 MJ. If operational restrictions on vessel size or vessel velocity are implemented, the impact energy can be reduced according to Figure 4. Alternatively, with a given capacity to tolerate impacts, the operational restrictions can be inferred. Unless a proven velocity limitation system is implemented (that a faulty DP system cannot override) a velocity of 3 m/s shall be assumed in head-on collision. It should be noted that the type of vessel may influence the energy absorption between vessel and structure, and shall be included in any such operational restriction evaluation.

The statistics in determining the relevant impact energy and velocity is based on an "all-vessel" evaluation. A possible reduction in velocity for vessels with DP3 systems have not been evaluated, and should be further investigated.

2.3 Flotel impact

Flotels moored close to an installation represent a particular collision hazard [15]. The probability of impact is governed by the probability of coming adrift in a critical sector, which again primarily depends on the probability of multiple mooring line failures. With a design of the mooring system, according to early design rules and practice, the probability of contact may be in the range $2 \cdot 10^{-4}$ to 10^{-2}, while it is probably less than 10^{-4} if the strength of the damaged mooring system satisfies the ALS requirements. In general the risk associated with flotel collision should be examined in each case where temporary or permanent use of such units is considered

2.4 Shuttle tankers

Tankers which are used for offshore loading may be involved in the following types of collisions:

- Collision of powered or drifting tanker with installation (FPSO) while in transit. This collision scenario can be treated in the same way as collisions of other passing vessels with platforms, by defining the tanker route as part of the shipping traffic data. This scenario is treated as a passing vessel collision event.
- Collision of shuttle tanker with FPSO during offloading. This may be due to a misjudgement or machinery failure

on approach or due to a mooring or Dynamic Positioning (DP) failure while in position.

The latter scenario is of interest here. The offloading operation can take place in different modes, e.g. alongside transfer and tandem moored transfer, according to the relative positions assumed by the FPSO unit and the shuttle tanker during offloading operations.

In the tandem mode, the shuttle tanker is moored to the FPSO by hawsers and/or Dynamic Positioning (DP), while the cargo is off-loaded through floating hoses (Figure 5). The vessels may also be subjected to significant motions in the horizontal plane, (surge, sway and yaw) due to waves and wind in harsh environments. In order to stay connected and maintain the separation distance, the shuttle tanker must position itself to follow the FPSO whenever it changes heading or position during offloading period, which on average may last more than 20 hours. Depending on the environmental conditions and the offloading procedures adopted, the shuttle tanker could be equipped with DP and/or assisted by tugs and/or a supply/standby vessel during offloading operations (approach, mooring, cargo loading and disconnecting mooring/ departure. DP is commonly used in the North Sea, but not in Africa, SE Asia and South American/Caribbean areas. One or more tugs or a support vessel usually helps avoid collision during FPSO mooring, offloading and disconnecting mooring.

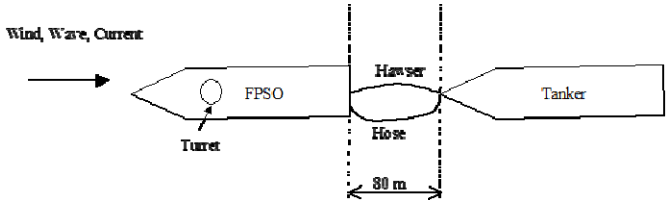

Figure 5 FPSO- Shuttle Tanker offloading operation.

The shuttle tanker operation can be summarized according to the following five operational modes:

1. Approach of the shuttle tanker to the FPSO system.
2. Mooring of the shuttle tanker to the FPSO system.
3. Connection of loading hose.
4. Cargo loading.
5. Disconnection of loading hose and mooring; and departure.

The risk in each of the operational modes need to be considered.

The major risk in offloading between an FPSO and shuttle tanker is collision, but, as incidents have shown, that hawser or hose breakage – that may cause oil spill and production delays, should also be noted. Last but clearly not the least, human factors play an important role during the incidents. This is reflected by human errors that initiate or escalate incidents, human intervention of crisis that avoided more severe damage, or organizational factors, e.g. tanker bridge resource management [16].

The impact energy in the bow-stern impact is found to be proportional to the travelled distance of the shuttle tanker before impact; and less than proportional to the mass (displacement) of the shuttle tanker [17]. In the period of 1995-2002 there were 20 incidents and near misses, wheras 5 of the events were classified as collisions, While the ratio between near-misses and incidents was 6:4 for DP1 systems, it wav 6:1 for DP2 systems. Moreover, the number of collisions corresponds to an annual collision frequency per facility of $1.0 \cdot 10^{-3}$ and $1.5 \cdot 10^{-3}$ in UK and Norway, respectively. It is interesting to note that the average annual collision frequency in the UK has been decreasing since 1997.

The following collision shuttle tanker/FPSO events have occurred in the North Sea in the period 1998-2014

- Shiehallion FPSO, 25.09.1998
- Norne FPSO, 05.03.2000 (impact energy: 31 MJ)
- Njord B FSU, 13.11.2006 (impact energy: 61 MJ)
- Sciehallion FPSO, 08.10.2009

Risk assessment of this kind of operation needs to be based on previous incidents involving FPSO/shuttle tanker contact. But experiences [18] with offloading with shuttle tankers from an articulated loading platform or buoy are also very useful.

Actually, based on observed collisions between shuttle tankers and loading buoys in the Norwegian sector, Kvitrud and Nilsson [19] estimated the frequency of such events to be $1 \cdot 10^{-3}$ per loading. Previous estimates indicate $1.4 \cdot 10^{-4}$ to $2 \cdot 10^{-3}$ per loading. If 100-150 off-loadings per year is assumed, the annual frequency is of the order 0.1.

Collisions are caused by an uncontrolled relative movement between the vessels, and are initiated by

- technical faults (DP positioning, reference system, main engine/propeller) which require countermeasures, and human response is inadequate
- erroneous DP operation in temporary modes; or when changes occur during loading operations due to weather changes, and especially when particular relative movement (surging and yawing – e.g. fishtailing) between the FPSO and tanker occur.

The human errors are due to operators erronous expectations of DP functions, in view of the relevant physical conditions, which lead to improper use of DP to position and manoeuvre the tanker. On the other hand, human intervention (evasive manoeuvring) right before collision is an important issue, as born out by the incidents experienced so far.

The implication for shuttle tanker and FPSO with 140 000 tons displacement, is that the speed corresponding to a travelled distance of 50 m is 1.16 m/s and after 100 m 1.62 m/s. This fact might imply that a smaller distance between the shuttle tanker and the FPSO is beneficial. This is only true, however, if no corrective/evasive action is taken [16].

Unless a more detailed risk assessment is conducted, taking relevant risk reduction measures into account, the minimum shuttle tanker bow impact on FPSO stern should be taken to be 70 MJ.

To reduce the risk of shuttle tanker collision in tandem offloading, among others the following measures are envisaged:

- Use of dedicated tankers with high degree of redundancy (twin engines, CPP etc.), bow thrusters
- Use dedicated crew, improve training
- Use stringent procedures for shuttle tanker approach on
- Improve reliability of machinery
- Improve reliability of GPS (use DARPS on FPSO)
- If not add thrusters on FPSO
- Assess dynamic interaction between tankers in different environmental conditions

An other collision risk reduction can be achieved by using the FPSO and shuttle tanker in a different configuration during the offloading [20].

3 DESIGN AGAINST ACCIDENTAL SHIP IMPACTS – THE ALS CRITERION

Accidental Collapse Limit State (ALS) criteria are introduced to limit the corresponding residual risk associated with accidental actions, i.e. to prevent progressive failure. The basic principle relates to the fact that accidents develop in a fault sequence of events and it becomes important to establish a barrier to stop the escalation of the accident. This goal could be achieved by [21,22] by either of the following approaches

- designing the structure locally to sustain accidental actions and other relevant simultaneously occurring actions. (key element design); i.e. a quantitative "ULS" approach for designing elements. This a component design check.
- designing the structure by "accepting" local damage but requiring the damaged structure (due to accidental actions) to survive relevant actions (alternate path design).
- designing the structure to meet robustness requirements through (prescribed) minimum levels of ductility, continuity and tying (Tie-force based design methods):In practice ductility and continuity are also crucial in making the second method work.

One approach could be to design the platform with sufficient local strength (and stiffness) to resist the ship impact (i.e. with energy absorption in the ship). For concrete structures the energy absorption is negligible and the design check becomes a strength check.

The second method was initially made a regulatory requirement by NPD [23] and is currently specified in NORSOK N-001 [1]. The initial ALS criteria, based on accidental actions with an annual exceedance probability of 10^{-4} were said to imply a (maximum) probability of total loss of 10^{-5}. (e.g. [4, 24]). In the following probability values used by NPD (now NORSOK N-001), are referred to, but other values might be used in other regulatory regimes.

The structural integrity criterion in NORSOK N-001 is expressed by a two-step procedure as illustrated in Figure 6, based on characteristic actions and resistances. The first step is to estimate the initial damage e.g. due to accidental actions like ship impacts with an annual exceedance probability of 10^{-4}. This exceedance probability refers to accidental events on the whole platform and needs interpretation, as discussed by Moan [4].

When the probability of impacts from different types of ships have been determined, the associated collision energy can be estimated from their mass and speed. The variation in speed can be accounted for by a conditional probability. An energy spectrum, showing the cumulative frequency of collision versus the collision energy can then be generated and the event corresponding to an annual exceedance probability of e.g. 10^{-4} may be determined. The most probable impact locations (bow, stern, side) and impact geometry should be established based on the dimensions and geometry of the structure (FPSO) and the impacting vessel, and should account for draught variations, operational sea-state and motions of the vessel and structure.

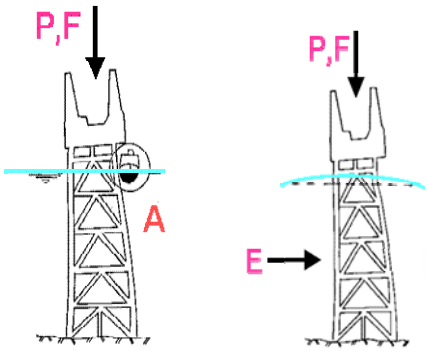

Figure 6 Two-step Accidental Collapse Limit State check of global failure, considering accidental (A), Environmental (E), functional (F) permanent (P) actions.

A fully integrated ship impact analysis is fairly demanding. It is, therefore, often found convenient to split the problem into two uncoupled analyses, namely, the external collision mechanics dealing with global inertia forces and hydrodynamic effects, and internal mechanics dealing with the energy dissipation and distribution of damage in the two structures. The result of the latter analysis is an estimate of the strain energy dissipated based on the load deformation characteristics of the ship and installation given in e.g. NORSOK N-004 (schematically illustrated in Figure 7), and by assuming that the forces at the contact point is in equilibrium ($R_s = R_i$ in Figure 7). The assessment of the load-deformation characteristics for the ship and installation, respectively, is based on the idealized assumption that the "other" object" is rigid. In the revised version of the DNVGL document [33] interaction effects are taken into account.

The second step is to demonstrate that the damaged structure resists - without global failure - relevant functional and environmental actions with a characteristic value depending on their correlation with the event initiating damage. The default characteristic environmental action in step 2 is defined by an exceedance probability of 10^{-2}. However, if the correlation between the environmental action and accidental action (ship impact) is low, the environmental action at an exceedance probability of 10^{-1} is applied. Action and resistance factors for steel structures are taken to be 1.0 in these design checks.

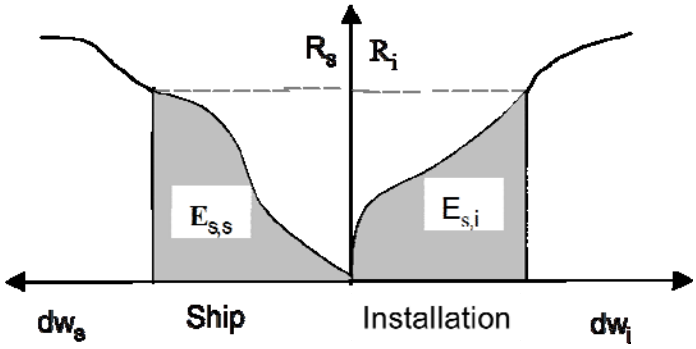

Figure 7 Dissipation of strain energy in ship and installation

It should be noted that for accidental phenomena where the intensity increases slowly with decreasing probability of exceedance, a ULS design check shall be carried out with a characteristic value of each accidental action which corresponds to an annual exceedance probability of 10^{-2} per installation as well as with appropriate load and resistance factors. The ALS design check should be carried out with a characteristic value of each accidental action which corresponds to an annual exceedance probability of 10^{-4} per installation.

The NPD/NORSOK approach is applicable to other failure modes, like a rigid body instability of a platform with damage e.g. due to ship impact, and the station-keeping system failure during an impact event.

To estimate damage, i.e. permanent deformation or rupture of parts of the structure, nonlinear material and geometrical structural behaviour need to be accounted for. While in general nonlinear finite element methods need to be applied, simplified methods, e.g. based on plastic mechanisms, are developed and calibrated by using more refined methods, to limit the computational effort required.

In connection with instability failure modes (capsizing) the use of a risk assessment approach to determine the accidental actions and the corresponding damage, is an extension of the conventional method based on prescribed flooding of 1 or 2 compartments since the damage will depend on the material and layout of the structure. Moreover, it is noted that damage to the "submerged" parts of a floating structure, leads to a change of the floating position which hence will influence the wave and current actions on the structure.

Another issue is how normal uncertainties are dealt with. More sophisticated probability-based approaches in which some sort of quantitative or semi-quantitative model is constructed are now being developed. The purpose of these models is to establish a given level of reliability in the structure, i.e., that the probability of failure is less than some defined target value. Such methods are not currently implemented in codes and standards, although EN 1991-1-7 [25] does contain an annex which sets out a probability-based framework.

4 ASSESSMENT OF SHIP IMPACT DAMAGE AND CONSEQUENCES

4.1 Methods

To demonstrate compliance with ALS requirements calculation of the damage due to accidental actions as well as the ultimate capacity of a damaged structural system is needed. To estimate damage, i.e. permanent deformation, rupture etc of parts of the structure, nonlinear material and geometrical structural behaviour need to be accounted for. Dynamic effects may be of importance for explosions and ship impacts. Recent advances in computer hardware and software have made nonlinear finite element analysis (NLFEM) a viable tool for assessing damage and system resistance for steel structures. Examples of general purpose computer codes, which have been widely used are ABAQUS, ANSYS and LS_DYNA. Specialised software, like USFOS, is available for particular tasks.

Simplified methods based on plastic analysis often provide fast and amazingly accurate estimates of the damages caused by accidental actions on steel structures (e.g. NORSOK N-004) and are especially useful in early design for screening purposes. In particular cases where simplified methods have not been calibrated, nonlinear time domain analyses based on numerical methods like the finite element method should be applied.

In finite element analyses of collisions a careful choice of mesh is required in order to obtain accurate results, especially for components deforming by axial crushing. A major challenge in NLFEM analysis is the prediction of ductile crack initiation and propagation. This problem is not yet completely solved. Crack initiation and propagation should be based on fracture mechanics analysis, using the J-integral or Crack Tip Opening Displacement method rather than simple strain considerations. The simplest approach to the problem is to remove elements once the critical strain is attained. However, deleting elements disregards the fact the large stresses can be maintained parallel to the cracks. An improved modelling is to introduce a double set of nodes such that the elements are allowed to separate once the critical stress is attained. A drawback with a double set of nodes is that the potential location of cracks needs to be defined prior to the analysis.

Compliance with the global strength requirement of the damaged structure, can in some cases be demonstrated by removing the damaged parts, and then accomplishing a conventional ULS design check, based on a global linear structural analysis and ultimate strength checks of components. Such methods may be very conservative, especially for damaged structures. Fixed platform analyses are carried out by modeling the pile-soil behaviour by equivalent linear or nonlinear concentrated springs or, distributed springs along the piles, or continuum (finite element) model that represent stiffness and foundation, capacity, appropriately using the material properties in the different soil layers. Soils exhibit nonlinear behaviour, even at low load levels, which needs to be accounted for. Software dedicated for progressive collapse analysis of frame offshore structures have also been developed [26] and implemented, e.g. in USFOS and SACS.

4.2 Case studies for different types of structures

4.2.1 General

In this section some examples to illustrate the consequences of ship impacts on various types of offshore platforms are presented. The purpose of the assessment of impact damage is to illustrate the effect on structural integrity and also possible damage that can cause flooding or oil leaks, pipe damage/rupture of hydrocarbon piping that lead to leak of oil and gas, especially for floating platforms. A particular aspect is what kind of strengthening would be needed to prevent or limit the local damage.

The impacting vessel is typically made of steel while the hit structure can be a steel or reinforced concrete structure. For impacts between steel structures the energy absorption in principle takes place in both structures. Sometimes it is assumed e.g. that an impacting bow structure is rigid and all energy is absorbed by the hit structure. For vessel impacts with concrete structures the energy absorption is commonly assumed to take place in the vessel and the force during impact is determined and applied in a strength check of the concrete structure. It is noted that concrete structures can be sensitive to a local high load causing shear failure or puncturing.

4.2.2 Jackets

The behaviour of a jacket designed with collision resistant braces in the water plane area was investigated by Storheim and Amdahl [27]. Figure 8 shows the finite element model.

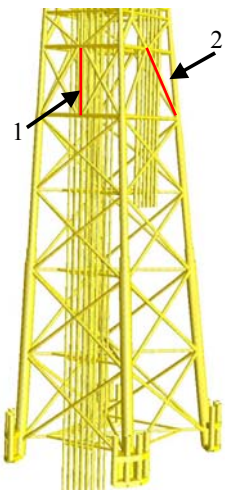

Figure 8 FE model of jacket with collision positions indicated.

The vertical brace in position 1 is 50 mm thick, has a diameter of 1.3 m and is 21 m long, as measured from the chord-to-chord intersection. The diagonal brace in position 2 is 60 mm thick, has a diameter of 1.2 m and a chord-to-chord length of 24.4 m. The braces are fabricated of high-strength steel with a yield strength of 460 MPa.

Figure 9 shows the force deformation curve for the bow colliding against the brace in position 1. The local deformation (indentation) of the cross section of the brace is about 0.5 m (40% of the diameter), and the global deflection is approximately 1.1 m. The bulb crushes with a maximum force of approximately 18-20 MN against a rigid brace. However, for a deformable 50-mm brace, the integrated crushing force exceeds 22 MN, because the brace wraps around the bulb during deformation. Consequently, the collapse resistance in bending, R0, should be at least 20 MN to avoid the development of substantial lateral, plastic deformations - including plastic bending and denting of the brace.

It was further found that that the energy absorption (and hence the damage in the brace), became negligible if the brace thickness was increased from 50 mm to 65 mm. This fact illustrates the rapidity of the transition from shared energy - to strength design.

Figure 10 shows the contact force versus the total ship displacement for a collision at position 2. Only the bulb is in contact with the brace. The brace crushes the bulb significantly but undergoes a global plastic deformation of 0.6-0.7 m before the bulb crushing force levels out. No significant local denting of the brace is observed. The contact force is smaller than for impact against a rigid vertical brace with a similar diameter. The contact length in position 2 is smaller than the contact length in position 1. In addition, the chord-to-chord length is larger in position 2 than in position 1, so it is reasonable to assume that the collision force is concentrated in position 2. Assuming a concentrated load, the plastic collapse resistance for the brace is 11.9 MN. This value compares well with the simulation results.

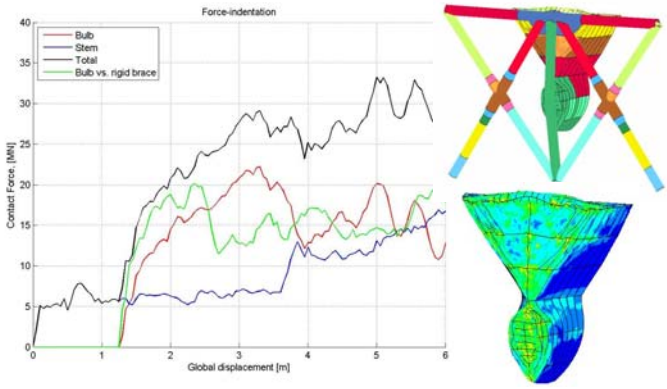

Figure 9 Force vs. deformation - position 1

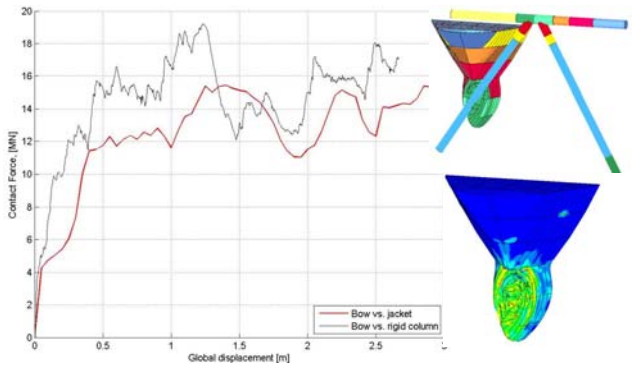

Figure10 Force vs deformation for a ship impact against a deformable and a rigid vertical brace in position 2.

4.2.3 Semi submersibles

Collision between a ship bow and stern corner of a semi-submersible column was studied by Storheim and Amdahl [28]. Approximately 1/3 of the front section of the column of a floating production platform was modelled, as shown in Figure 12 a. The column width is 17 m , the decks in the column have a plate thickness of 12 mm and the ring frames (T1100x300/12x20) are spaced approximately 3 m apart. The vertical stiffeners are typically HP 300 x 12 with a spacing of 0.65 m. In the rounded corners of the column, the outer shell stiffener spacing is approximately 800 mm. The net shell plate thickness is about 16-17 mm in the collision region. All degrees of freedom of the rear of the modelled section are fixed. The element size is approximately 100-120 mm, which corresponds to a minimum of three elements over the stiffener web and five elements in the plate between each stiffener. Herein, only results for bow impacts are included

Figure 11b shows the FE models that were established for the bow and stern of a modern 7500-ton displacement offshore supply vessel. The main dimensions of the vessel are as follows: an overall length of 91 m, 79 m length between perpendiculars, a breadth of 19 m, a molded depth of 7.6 m and a scantling draught of 6.2 m. The bow model has a general element size of 120 mm. The plate thickness varies from 7 mm for the decks to 12.5 mm in the bulb. The stiffener spacing is approximately 600 mm, with ring stiffeners and breast hooks of approximately 250x15 mm in the bulb. HP220x10 stiffeners are used in the forecastle structure. The bulb radius along the centerline is approximately 2 m and 1 m along the stringer deck in the middle of the bulb.

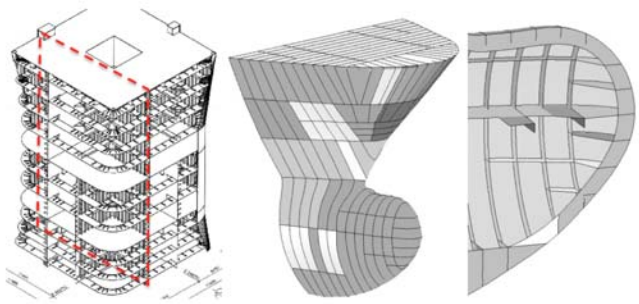

Figure11 Semi-submersible column (a) and Supply vessel bow model (b)

Figure 12 shows the simulated damage for a bow impact against the corner of the column, with the bulb impacting a ring frame. Figure 13 shows the contact force, which is split into a stem contribution and a bulb contribution, together with the force-displacement relationship for a bow against a rigid column with a 5-m radius that is similar to the platform corner radius. The stem force-deformation curve is close to that obtained for crushing against a rigid column with the same diameter as the platform column.

In the initial phase of the collision, the bulb is stronger than the platform, and the force-displacement relationship is thus governed by the strength of the platform column, as can be seen from the gradient of the force-deformation curve being lower than that for rigid column impact. When the membrane strength of the platform shell is mobilized, the bulb starts to crush at a peak force of 22 MN.

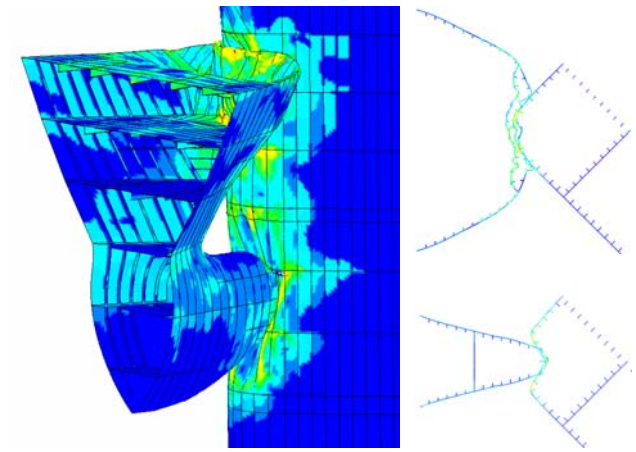

Figure 12 Simulated damage for the bow and column after 51 MJ energy dissipation (corresponding to 4 m global displacement in Figure 13.

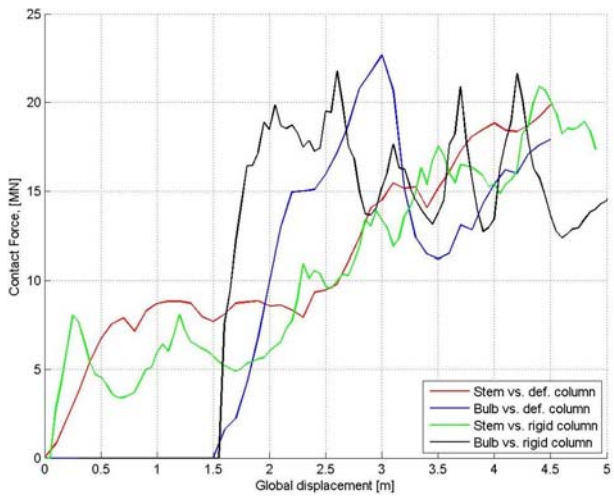

Figure 13 Force vs. total displacement of a deformable bow for impacts against the corner of a deformable and a rigid platform column.

The failure model predicts fracture of the outer shell shell when about 37 MJ of strain energy has been dissipated. However, fracture may occur earlier at the hard points not included in the model (anchors, bollards, etc.), as was observed in the accident investigated by Pettersen et al. [29]. This and other analyses carried out by the authors show that the bow forecastle tends to be weaker than typical platform columns and will absorb considerable energy by crushing. Any platform damage caused by the forecastle will also take place significantly above the water line.

The critical issue is penetration of the bulb into the column, which would lead to flooding. For bulb collisions between ring frames or on a ring frame for typical platform columns penetration is likely to take place. When the impact is below the water line, this scenario will in the first place lead to flooding of a single compartment. Storheim and Amdahl [27]

found that the bulb of a conventional vessel did not cause fracture of a platform column when contact took place on a vertical bulkhead, but penetration and flooding of two tanks could not be ruled out taking into account modelling and material uncertainties. Impact with the bulb of a lightly ice strengthened bulb (DNV ICE 1C class) altered the strength ratio significantly, and penetration of the bulb into two compartments was very likely. A three-compartment damage could not be ruled out either, because the forecastle was slightly stronger than that of the conventional vessel.

Collisions on the intersection of a vertical bulkhead and a horizontal deck can potentially flood four compartments and would impair damage stability requirements for most platforms. The platform will be stronger at such locations, and the tendency to crush the bulb will increase. No systematic study of this scenario has so far not been presented in the literature.

By increasing the dimensions and/or material strength of the column structure in the areas exposed to bulb impacts it is feasible to obtain strength design as shown by Storheim [30].

4.2.4 Jack-ups

Jack-ups supported on the seabed in their operational mode, have a high fundamental eigenperiod, up to 8 seconds in the North Sea. Thus, the response to high-energy collisions will take place in the impulsive – or the dynamic domain. This calls for nonlinear, dynamic analysis because the temporal impact-force history will depend on the jack-up response and cannot be calculated a priori on the basis of design curves for the impact force vs. deformation of the ship bow as mentioned above. In such analyses the design impact force curve for the ship may conveniently be represented by a nonlinear spring.

Amdahl and Holmås [31] investigated the response of a jack-up in 100 m water depth for a 67 MJ supply vessel bow impact and found that the platform and ship absorbed 30 MJ and 12 MJ respectively by plastic deformation, while as much as 25 MJ was absorbed as elastic energy in the platform and was restored as kinetic energy after the impact. The behaviour of the tubular joints was not studied in detail, but they concluded that the compliance of jack-ups is definitely positive with respect to survival of high-energy impacts

4.2.5 Ship shaped structures

Collision between supply vessel and FSU was studied by Hong et. al. [32]. The FSU was a converted tanker. The supply vessel size was 6500 ton, but a 2200 displacement vessel was also considered. Three different scenarios were analysed. Scenario 3 represents the tanker in fully laden condition and the supply vessel in ballast condition or with significant trim/heave-pitch motion. The forecastle will only be in contact the FSU side after a significant bulb deformation or penetration. In In Scenario 2 the FSU is in ballast condition, and the forecastle deck is the first to contact the FSU side.

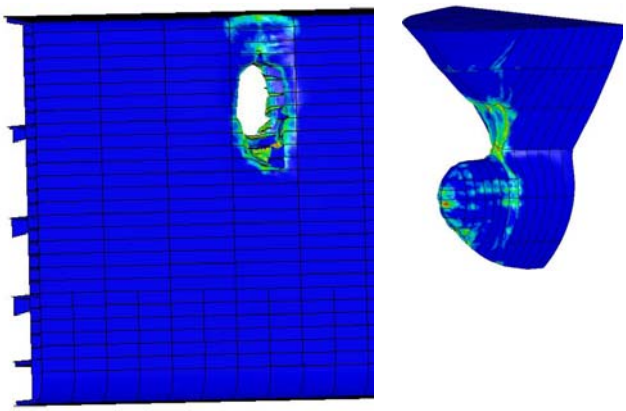

Figure 14 Damage in ship side after ship bow impact in Scenario 3.

Some results are summarized in Table 3. As expected, Scenario 3 is most critical, because the bow forecastle will be engaged to the least extent. The bulb penetrates the inner side with little deformation of the bulb and some energy dissipation of the side. The total energy dissipation for penetration of the inner side is 22 MJ (Fig. 14). The energy dissipation in the FSU side is 19 MJ, and only 3 MJ in the bow. The energy corresponds to 2.5 m/s or 4.9 knots for 6500 tons displacement.

Apparently, the bulb of the 6500 tons vessel does not penetrate the side in Scenario 2. Taking into account the uncertainty and sensitivity of the simulation with structural configuration of the bulb, this cannot be relied on. If the bulb penetrates the side the critical energy is estimated to be at least 60 MJ, corresponding to 8 knots

Table 3 Critical energy and striking speed for penetration of inner tank wall

	6500 ton		2200 ton		2200 - extreme	
	Energy [MJ]	Speed [kts]	Energy [MJ]	Speed [kts]	Energy [MJ]	Speed [kts]
Scenario 1	32	5.8	40	11.2	20	7.9
Scenario 2	>90(60)	>9(8)	60	13.7	40	11.2
Scenario 3	22	4.9	29	9.5	17	7.3

4.2.6 Shuttle tanker bow impact on FPSO stern

A high energy bow impact on the FPSO stern by the shuttle tanker (Figure 5) might lead to damage to the flare tower and penetration into the engine room and even the helicopter fuel tank. It is considered highly improbable that the collision will threaten directly the overall integrity or stability of the FPSO. The consequences of such a scenario are estimated to be limited to local damages such as [17]:

- Rupture of FPSOs ballast tanks and engine room located at the stern
- Rupture of the fuel tanks located at stern of FPSO
- Damage to the living quarters

Such collision events may be caused by shuttle tanker drift or drive off. It is noted that penetration of the machinery room which causes flooding, may require 10-100 MNm depending upon the relative size of the FPSO and the shuttle tanker, their

design and the material used. Less severe impacts may lead to collapse of the flare tower and cause fire and production stop.

4.2.7 Other collision scenarios

Collisions against the decks of semi-submersibles have so far been considered to a limited extent. However, with increasing supply vessel size this scenario becomes more likely. It is noted that such a collision scenario could have occured in the Fars Symphony-West Venture impact in 2004 [29] had the circumstances been slightly different. Collision between supply vessel superstructure and semi-submersible deck was simulated for two impact scenarios; the platform in operating draft and in survival condition. In operating condition a large part of the superstructure will be in contact with the deck, in survival condition only the wheel house will hit the deck.

Figure 15 illustrates the damage for 50 MJ and 290 MJ energy dissipation. Damage to the platform deck is moderate; with an impact force of 40 MJ at the end of the collision. Presumably, the collision could have more serious consequences for the pilots. The extreme damage corresponds to impact from a vessel of 9000 tons travelling at a velocity of 8 m/s.

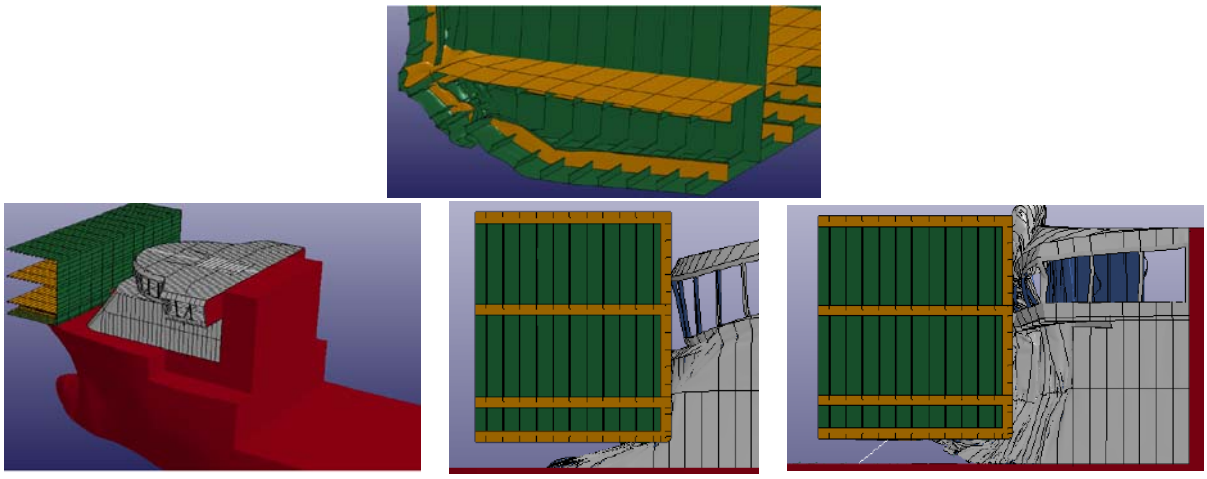

Figure 15 Collision against the deck of a semi-submersible in operating condition.

5 CONCLUSIONS

The collision risk should be estimated and controlled by reducing the probability of collisions and by design reducing the consequences. Due to the high frequency of severe bow impacts by attendant vessels and shuttle tankers in tandem off-loading modes it was suggested, based on the research presented in this paper, to implement minimum requirements which need to be followed unless an acceptable risk can be documented by a quantitative risk assessment based on operational measures relating to the vessel size and speed. This approach has now been implemented in the revised version of the Norsok N-003 standard [34]. .

For typical North Sea jackets it will be difficult to design braces such that a single brace will absorb the required energy for standard bow collisions. A viable alternative may be to design the braces strong enough to penetrate and crush the bow of the impacting vessel, so that the vessel dissipates most of the energy. To achieve this goal the brace must have sufficient plastic bending capacity and comply with compactness criteria with respect to local denting. Detailed requirements will be implemented in a revision of DNV-GL Recommended Practice for design against accidental loads [33].

Jack-up structures for North Sea conditions (water depth approx. 100 m) are relatively compliant and may absorb a significant part of high collision energies by elastic deformations. However, the ship bow should preferably absorb energy. The braces should therefore comply with plastic bending and denting requirements so as to penetrate the bow. For both jackets and jack-ups the ultimate strength of the adjacent tubular joints should match the capacity of the brace.

For bow collisions against stiffened columns of semi-submersibles and the side of ships shaped units, the forecastle deck for standard support vessels will be crushed with no – or moderate strengthening of the side shell plating and/or stiffeners.. However, extra strengthening is required if the platform side shall be capable of crushing the bulbous part of the bow. Penetration by a bulb below the water line, is more critical than penetration by the forecastle because flooding of void spaces is unavoidable and hydrostatic stability may be impaired.

A large variety of bow shapes have been developed.in recent years. In addition, some vessels may have additional strengthening to facilitate navigation in ice conditions. If strength design (most energy absorbed by vessel) shall be assumed it is imperative that the ship deformation curve adopted for the design is representative for the bow configuration of the support vessels that will service the platforms during operation. The sheer increase of the support vessel size that has taken place during the last two decades and potentially in the future may render new collisions scenarios that may need specific design considerations.

Further work should be directed to assess the risk of attendant vessel impact as a function of the attendant vessel traffic; as well as the impact of shuttle tanker on the FPSO in tandem offloading scenarios.

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7.3 Presentation by Himanshu Singh

NTNU Norwegian University of Science and Technology

International Association of Oil & Gas Producers

Non linear dynamic analysis of blast loading on topsides structures

Himanshu Singh, Kok-Leong Toh
Shell Global Solutions International BV

The 3rd Offshore Structural Reliability Conference
OSRC2016
14-16 September, Stavanger, Norway

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Reserves: Our use of the term "reserves" in this presentation means SEC proved oil and gas reserves.

Resources: Our use of the term "resources" in this presentation includes quantities of oil and gas not yet classified as SEC proved oil and gas reserves. Resources are consistent with the Society of Petroleum Engineers 2P and 2C definitions.

Organic: Our use of the term Organic includes SEC proved oil and gas reserves excluding changes resulting from acquisitions, divestments and year-average pricing impact.

Shales: Our use of the term 'shales' refers to tight, shale and coal bed methane oil and gas acreage.

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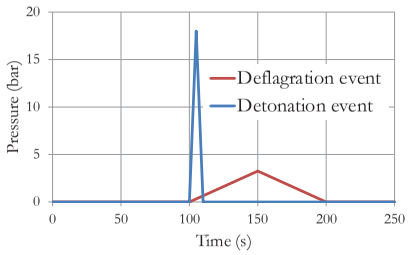
Outline

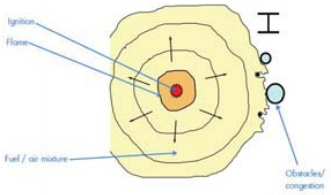
- Detonation & Deflagration
- Explosion risk assessment
  - Detonation
  - Deflagration
- Comparison with theoretical solutions
- Conclusion

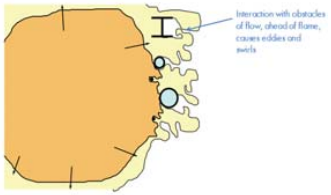
Detonation and Deflagration

- Deflagration and Detonation events
- Most vapour cloud explosions in the oil & gas industry are considered to be deflagration –
  - Deflagration has a lower peak over pressure but longer duration
  - Detonation has a higher peak over pressure but a shorter duration
- Detonation to deflagration transition happens at around 8 bar overpressure at the source

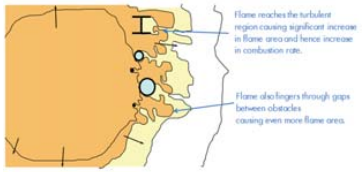


Congestion

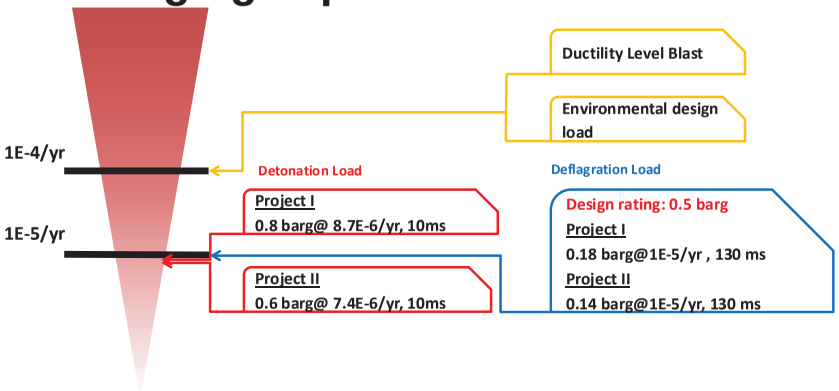




- Congestion increases gas build up volume.
- Cause turbulence effects, increases gas burning rate.



Managing explosion risk



- $1E^{-4}$/yr – Items whose failure could lead to escalation to more than one block of module
- $1E^{-5}$/yr – Items whose failure could lead to direct impact to TR

Explosion risk assessment detonation

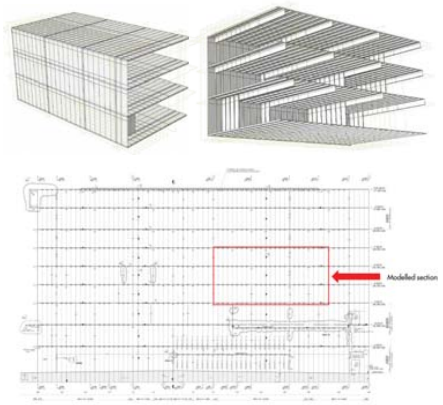
- For an offshore installation, during a major hazard event, the primary aim is personnel protection, where personnel is located in the Living Quarter
- Critical to ensure Living Quarter is not impair due to blast loading, i.e. structural integrity, smoke and gas tightness, etc.
- For this case study, DDT is observed, generated a blast pressure load of ~1 (0.7) bar though with a very short duration as compared to a deflagration event.
- The design rating for the Living Quarter blast wall is 0.5 bar.

Steps

- Develop FE model
- Apply blast loading as pressure load
- Carry out static pushover assessment to establish the resistance
- Carry out dynamic assessment
- Compare the result with analytical models

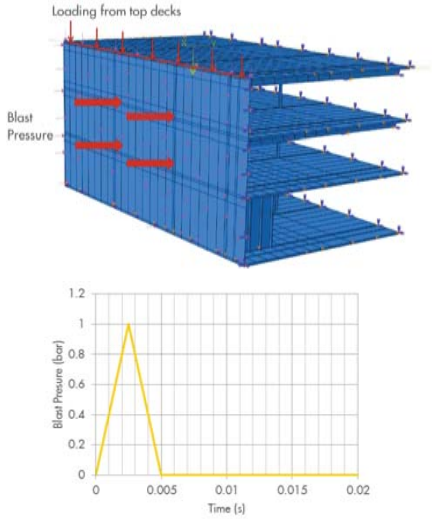
FE model

- A representative section of the Living Quarters was modelled
- LQ wall is modelled with Shell elements with appropriate wall thickness
- Wall modelled as elastic fully plastic material



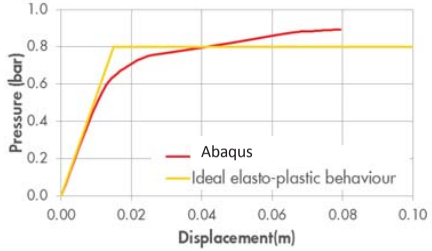
Loading

- Loading is applied on the middle panel as uniform pressure
- This has been done to reduce the boundary condition effects
- Loading from the top decks has been modelled as uniform distributed load
- The blast loading for a detonation event on the LQ wall is shown in the figure



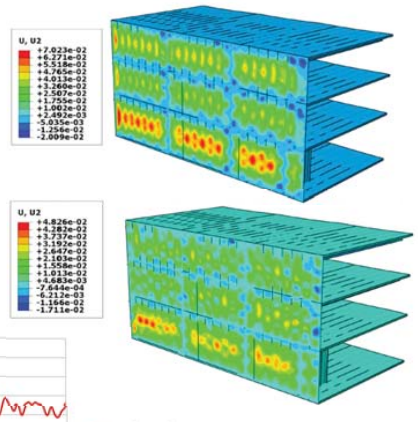
Static Pushover assessment

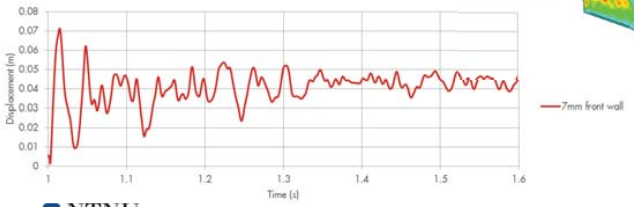
- Blast Pressure is ramped up on the wall until the collapse is observed
- Figure on the right demonstrates the load displacement of the central node of the middle panel along with Pressure
- A typical Elasto-plastic behavior is observed
- Analysis shows that the modelled wall can resist up to 0.8 bar before collapsing



Dynamic assessment

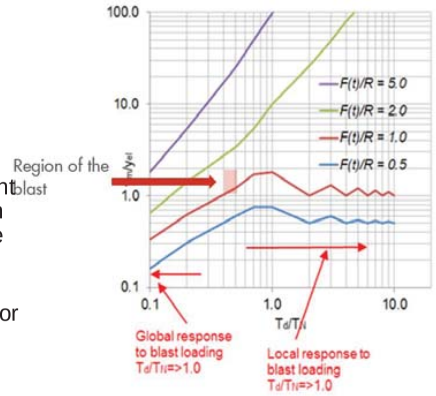
- This analysis is carried out in two steps-
  - Step 1 Gravity is applied
  - Step 2 Blast loading of 1 bar is applied
- Owning to the very short duration of the blast minor deformation is observed as shown in Figure below





Comparison with theoretical solution

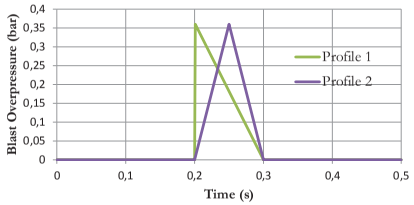
- The structure can resist a blast loading of 0.8 bar
- For dynamic assessment 1.0 bar pressure has been applied
- Hence F(t)/ R = 1.2
- Based on the dynamic assessment the natural period of the wall is Tn = 0.0125 s and the duration of the blast loading (Td) is 0.005 seconds, hence Td/ Tn = 0.3
- Based on analytical solutions minor plasticization is expected which corroborates with the results obtained from detailed dynamic assessment

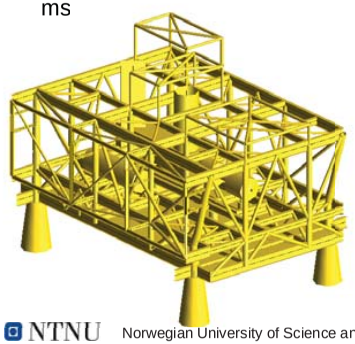


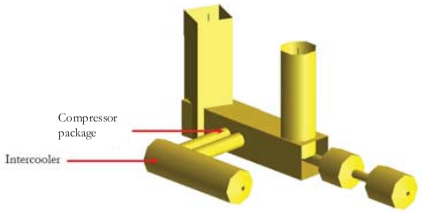
Modified Biggs Maximum response of Elasto-plastic one degree systems (undamped) due to equilateral triangular load pulses

Explosion risk assessment Deflagration (Gas Turbine package)

- Deflagration loading has a shorter impulse pressure but a longer duration
- A blast pressure of 0.36 bar is envisaged with a duration of 100 ms

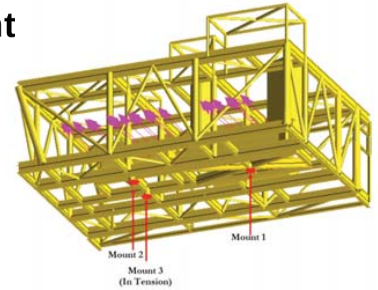




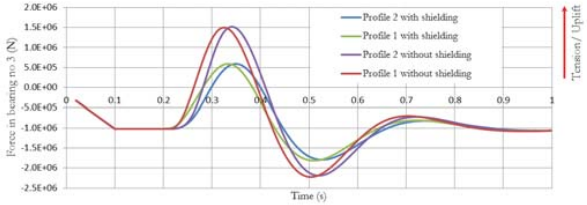


Loading and assessment

- The Gas Turbine is placed on a 3 point mount base skid on the module as shown in the Figure
- Blast loading is applied on the projected area of the gas turbine



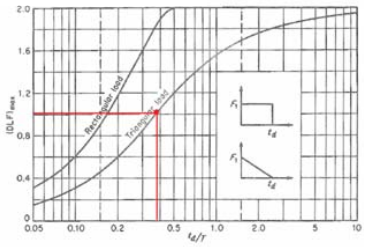
Gas Turbine mounts

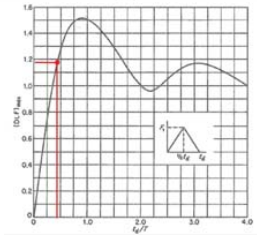


Forces observed in the mount in tension

Comparison with analytical model

- The duration of loading is 100ms
- Third mode of vibration (rocking mode) has a period of 342ms hence Td/T = 0.38
- As the duration of loading is close to the natural period amplification of loads is expected due to dynamics
- The amplification is observed in the dynamic assessment carried out





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Conclusion

- A detailed dynamic assessment of structural response to a blast loading benchmarks well with analytical solution
- The duration of the blast and impact pressure both have a significant influence on the structural response
- It is advised to carry out a detailed dynamic assessment to ensure structural integrity against the expected blast scenario

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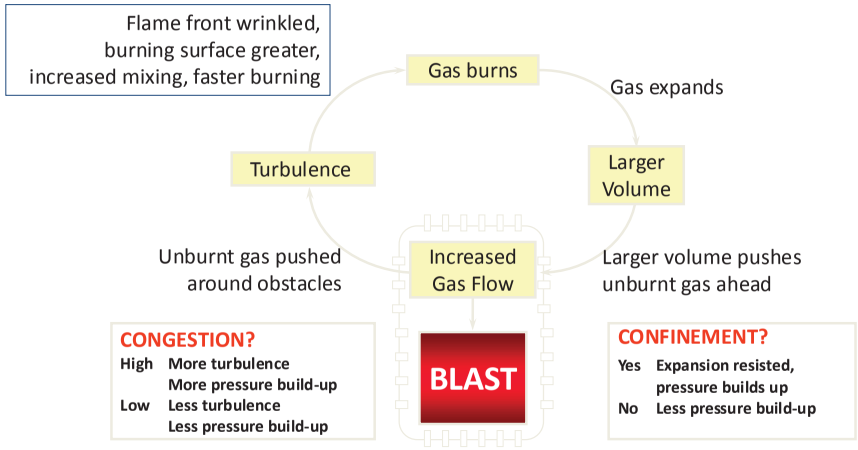
Back up Schelkin mechanism



Chapter 8

Session 6: Reliability of Concrete Platforms

8.1 Presentation & article by Pascal Collet

NTNU Norwegian University of Science and Technology

International Association of Oil & Gas Producers

Reliability of a concrete floating barge NKP case

Pascal COLLET – TOTAL E&P

The 3rd Offshore Structural Reliability Conference
OSRC2016
14-16 September, Stavanger, Norway

Content

- Concrete in marine environment
- N'Kossa Floating Production Unit Overview
- Risk Based Analysis on NKP
- NKP reliability assessment
- Way forward

Existing Floating structures

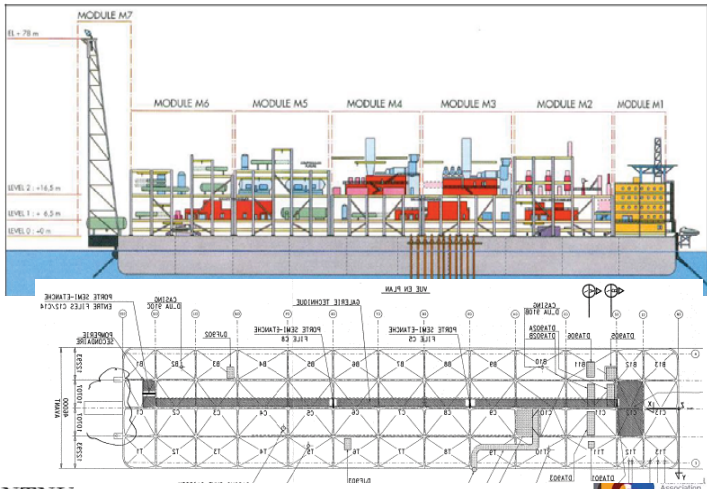
- From Ekofisk to Hebron
- From Ardjuna LPG Barge to Monaco Floating Dike

Gravity Based Structures

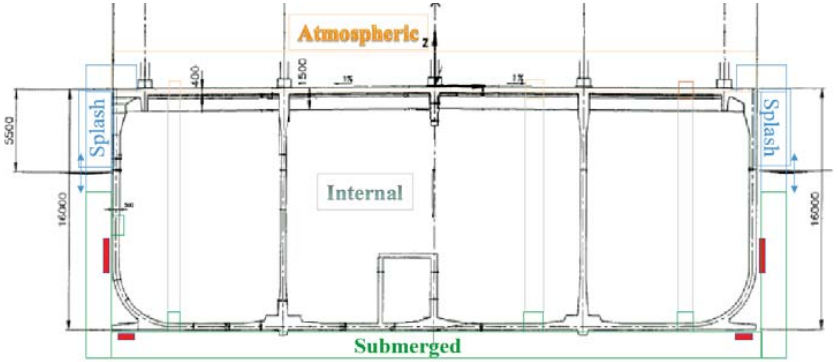
- Concrete caissons
  - Bridge footings
  - Breakwaters
  - Quay walls

NKP overview

Typical FPU



NKP Zones of interests



NKP Key figures

- **High Performance Concrete BHP 70:**
  - 27 000m3 (425Kg/m3 cement CPA/HP/PM, 35 Kg/m3 Silica Fume et 165 Kg/m3 Water, E/C=0,37)
  - Initial resistivity 55 Kohms·m
  - Permeability < 10^{-12} m/s
- **Passive steel:**
  - 5000 tons HA Fe E 500 / Mild Steel 235, 185 Kg/m3
- **Cathodic Protection:**
  - 209 anodes
- **Prestressing steel:**
  - 2350 tons, Tensile strength 1860 Mpa, 87 Kg/m3

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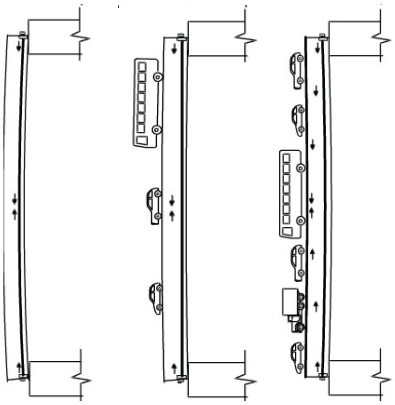
Prestressing concrete?

- Applied forces/loads to change the material behavior



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Construction

Croix (intersection)

Anodes Sacrificielle

Forme 10 pendant la construction

Sortie de la Forme 10

Bouchain avec quilles

Pose de la torche

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Deterioration processes

INTERNAL PROCESS

- Effect of sea water on cements (sulfate, chloride)
- Lime leaching-carbonation
- Alkali aggregate reaction
- Increase of permeability ⇒ Chloride Ingress
- Fatigue, Creep and Relaxation
- Corrosion of Steel Reinforcement

EXTERNAL PROCESS

- Accidents (Blast and Fire)
- Boat impacts
- Dropped object
- Corrosion of steel inserts and structures

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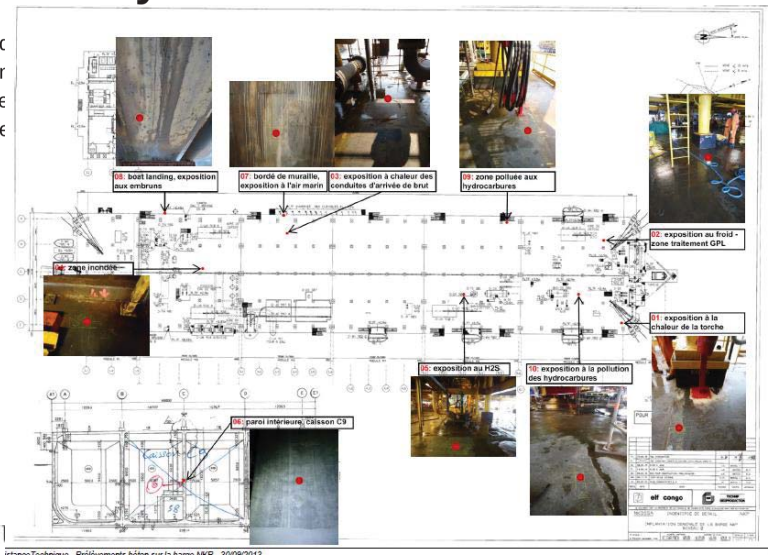
NKP – Built in Marseille

20 years old – What happened?

- NKP barge is in continuous production since 1996.
- Works on the topsides supports were performed in 1999.
- Cantilever support of the flare was refurbished in 2009 .
- The collision of a supply vessel in 2010 created a local damage in the aft starboard side shell and repaired in 2011.
- In 2013, a reclassification of the unit by Bureau Veritas starts.
- NKP will be operated more than 40 years within Floating Unit Integrity Management System (Project design life: 30 years!).

Laboratory Tests

- Chloric
- CO2 Ir
- Sulfate
- Sulfure



Concrete structure durability issues

- Materials
  - Achieved Construction Quality
    - Concrete cover, curing, prestressing works, etc.
  - Concrete mix design
    - Low porosity, compactness, compression stress, air content, etc.
  - Protective measures
    - Stainless steel, concrete hydrophobation, cathodic protection
- Design and operation
  - Loads and situations (change of weather condition?)
  - Cryogenic, Fire, Blast and Impact loads (not well known)
  - **Prestressing relaxation and creep of concrete**
  - Maintenance and monitoring

Risk Based Analysis

- Functional Analysis
- Failure scenario and probability of occurrence and consequences => risk level.
- Four main categories of failure :
  - natural material ageing,
  - marine environment impact,
  - accidental causes,
  - and normal industrial process.
- Visual inspection reports provide first assessments of hull integrity and guidelines for ageing processes.

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Risk rating

- Consequences of failure taking into account 3 aspects:
  - global structural integrity of the hull,
  - employee/public safety
  - and production loss.
- Probabilities of occurrence based on experience feedback on concrete structures in marine environment, accident rate for the hull or near - by structures and calculation results
- Material ageing – especially tendons' relaxation and concrete creep - for 30, 40 and 50 year old structure

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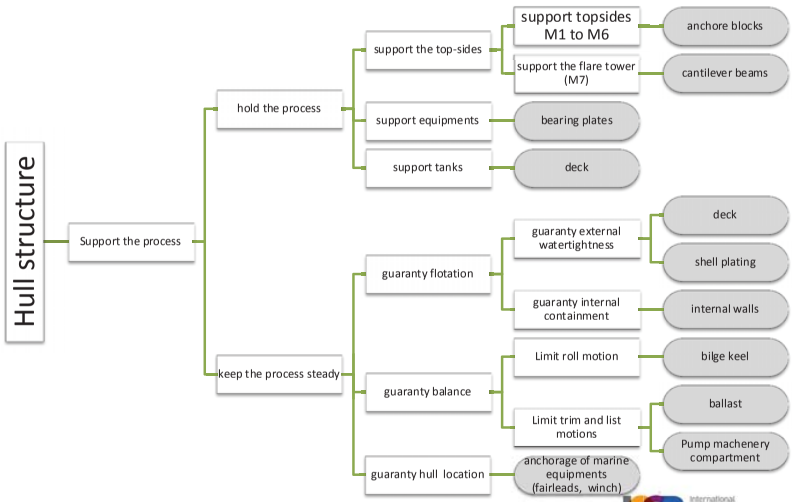
Fonctional Analysis



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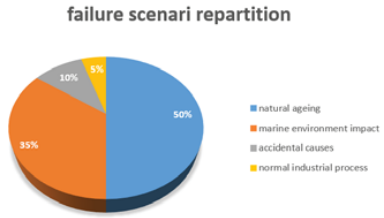
Over 3000 scenarii established

- Examples of areas requiring particular attention are:
  - Bilge keels and bottom penetrations
  - Topside stools and connection to deck
  - Flare tower support
  - Ballast tanks and "piscine"
  - Concrete cover of pre-stress cables heads at the splash zone
  - Mid ship caissons

	Risk repartition per year (%)		
barge's age	30-Year	40-Year	50-Year
acceptable	77.05	66.46	55.62
tolerable	18.04	26.7	30.71
unaceptable	0.61	2.54	9.37



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NKP Reliability assessment

- After 20 years, no huge maintenance had been required
- Missing data on stress in concrete, so prestressing state, oblige to use only visual inspection (crack analysis).

- Monitoring will provide data to update this assessment
- Risk Based Analysis provides an Inspection plan linked with monitoring system that provide tools to manage hull integrity for the next 20 years (postpone end of service life of more than 10 years)

- NKP reliability is closely linked to the concrete quality (first corrosion barrier) and quality management with company involvements in the project, contractors quality system, engineering survey and third party review.

Way forward

- Civil engineers and researchers are working to provide models, materials and procedures to improve structures older than 100 years but, where for Oil&Gas industry operation during 30 or 40 years, that will be already great to operate without hull painting activities…
- ISO 19903 update on going will address floating concrete structures
- PIANC guideline will be published soon as well and will provide interesting topics to improve durability and reliability of marine concrete structures

The 3rd Offshore Structural Reliability Conference
OSRC2016
14-16 September, Stavanger, Norway

Reliability of a concrete floating barge – the NKP case

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email: pascal.collet@total.com,

ABSTRACT:
N'Kossa barge is the first and unique concrete floating hull in off shore Oil&Gas industry - 220 m x 46 m x 16 m, displacement 107,000 t including 73,000 t for the hull and 34,000 t for the topsides - this proceeding address the philosophy followed by TOTAL to assess the reliability of the concrete hull.

After 20 years Congo off shore, TOTAL E&P Congo intends to develop and operate for another 20 years or more new assets connected to NKP process/utility plant. Initial hull design was done with a projected 30 years service life and the Operator faces now the issue of assessing the hull soundness and reliability for another 20 years.

Reliability of concrete barge is linked to hull integrity all along the life of the asset. Other aspects such as mooring, motion in wave, fire and blast response are also a part of barge reliability but not addressed in this paper.

A Risk Based Analysis was performed taking into account prestressed concrete behavior with all concrete issues and loss of prestressing strength, fatigue effects, material ageing in marine environment and potential accident such as boat impact, chemical leak or fire and blast, in order to assess the potential failure modes and the associated symptoms. Based on the results, monitoring is implementing to provide a hull assessment tool to extend hull useful service life.

KEY WORDS: Concrete Floating; Structure; Reliability; Monitoring.

1 INTRODUCTION

Floating Production Unit (FPU) N'Kossa Platform (NKP) is a unique concrete flotation structure, reliability is addressed by TOTAL by monitoring system implanted on concrete hull. In Oil&Gas context – continuous operating with hydrocarbon risks, in open sea where inspections and maintenances are much complicated than for other major structure on earth, a risk analysis was done to design the hull monitoring. This action is done after 20 years of operating without major trouble but also without monitoring data of the hull.

2 NKP OVERVIEW

The FPU was built in Marseille (France) in 1994-95 (Fig.1) and installed in 1996 in the N'KOSSA oil field in 170 m water depth some 60 km offshore Congo.
This is the first FPU and second concrete barge for O&G ever installed (Ardjuna Field –Indonesia - in 1974).
Its main characteristics are 220 m x 46 m x 16 m, displacement 107,000 t including 73,000 t for the hull and 34,000 t for the topsides. This concrete barge used in its construction 27,000 m3 concrete, 2,350 t pre-stressed steel and 5,000 t passive steel.

Fig. 1: FPU "NKP" in Dry dock Marseilles

All production facilities and living quarters for 172 persons are placed along the 10,000 m² deck surface which was subdivided into six modules: accommodation and central control, utilities, electric power generation, gas compression for re-injection, crude oil, and gas. Design production is 120,000 b/d of oil sent to the shore terminal and 1,300 metric tons/day of Liquefied Petroleum Gas sent to a 80,000 cu.m LPG FSO. The unit is maintained on position about 70 meters away of NKF2 platform by means of 12 mooring lines.

3 DESIGN AND MATERIALS

The hull is divided in 26 lateral spaces in 3 rows. Midship section is given on Fig. 2. Central void spaces are crossed by the "Technical gallery" which connects the aft and fore zones. A water tank is located in the capacity C10 to collect the water pumped from a depth of 50m. The water is sucked in with a Concrete pipe (metallic outside and concrete inside) that penetrates the side shell and runs through two capacities.

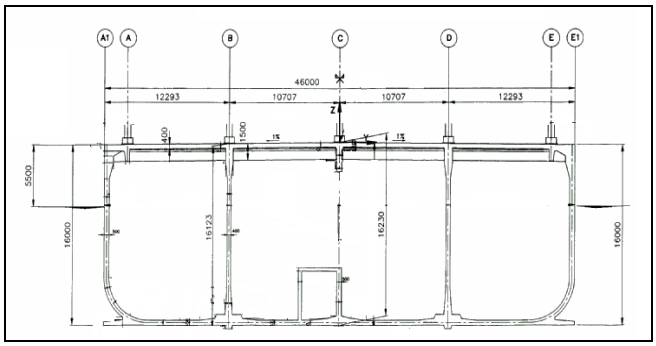

Fig. 2: Midship Section of the N'KOSSA concrete FPU

A high performance concrete was used for construction due to its qualities such as flowing capacity for pouring; resistance - 70 MPa - and low porosity once dried out. Low porosity in particular is essential to reduce phenomena like carbonation and chloride ingress. Ref [1] provides all details on structural components of the concrete shell.

The main idea is that the concrete hull is pre-stressed in 3 directions in order to maintain concrete continously under compression under the design forecasted external loads. Longitudinal tendons, i.e. longitudinal bending moment from prestressing, were arranged from design in opposition to the hogging still water bending moment of normal operation. The structural analysis has verified that such design construction keeps the overall structure in compression (no tension in section). Pre-stressing the structure in the three directions provides an assurance for water tightness and avoiding tension anywhere within the concrete section. Steel tendons are protected by grout in ducts, closely verifying that no water or air gaps are trapped within.

Reliability in such kind of concrete structure is strongly linked to losses of pre-stressed and not to the concrete.

4 PAST OF CONCRETE STRUCTURES

More than 50 concrete structures were built in Oil&Gas field from 1973 – Ekofisk, Philips, North sea, DORIS design – to 2016 – Hebron, ExxonMobil, Canada, Kvaner design.

In harbor design, as soon as water depth is over 20m, concrete caissons are usually considered as the right solution (Açu Port - Brazil, Breakwater Kamaishi - Japan, Tanger Med dike, Valencia harbor extension - Spain, etc.). Based on NKP feedbacks in 1996, Monaco Floating dike in 2000 had been designed and built with 100 years project life hypothesis. See Ref [2]

5 RELIABILITY ASSESSEMENT

5.1 Floating Unit Integrity Management System

NKP is part of Floating Unit Integrity Management System, references [3] & [4], and it was an efficient and economic way to handle, on a long-term basis, such highly rare floating asset. In this kind of program, inspection plan to assess the hull integrity if mandatory. After several years of practice, it appeared clearly that visual inspection doesn't full fill TOTAL and AIAC's expectations regarding integrity but also anticipated maintenance plan and TOTAL had added on its FUIMS program a monitoring system.

5.2 Structural Health Monitoring

In order to determine the most significant indicators that have to be followed to ensure a reliable Structural Health Monitoring (SHM) of the hull, a risk base analysis (RBA) was performed on the whole concrete structure. The method used was inspired by the Failure Modes and Effects Analysis (FMEA). The choice of this RBA method was driven by different factors such as the structure characteristics, its functions and its interaction with the environment.

5.3 Risk Base Analysis

First, a Functional Analysis (FA) was performed in order to determine the main elements that compose the hull associated with their function in the structure. The different elements were then cut into smaller parts (sub-elements) with unique characteristics (building material and design) and similar external interactions (environmental, factory process or accidental factor). For example, the deck was divided into 64 different sub-elements in order to take into account the differences in the design (for instance area of bearing) and the factory's equipment (tank, riser, pipes, winch, etc.) that can introduce various risks (extra load, potential fire, heat, chemical spill, etc.). For each sub-element, the different failure modes were determined as well as their origin and the associated symptoms. The results show that the established failure scenari can be sorted in four main categories following their causes: natural material ageing (50%), marine environment impact (35 %), accidental causes (10%), and normal industrial process (5%). Over 3000 failure scenari were established and for each of them, the probability of occurrence and the consequences were evaluated in order to calculate the risk level. The consequences of failure were evaluated taking into account three aspects: global structural integrity of the hull, employee/public safety and production loss. The probabilities of occurrence were evaluated based on experience feedback on concrete structures in marine environment, accident rate for the hull or near-by structures and calculation results (determination of the most stressed areas under ordinary swell – one-year return period – and exceptional swell – 100-year return period). In order to take into account the material ageing – especially tendons' relaxation and concrete creep, the different probabilities were evaluated for a 30-year old structure, a 40-year old structure and a 50-year old structure. This enable TOTAL to know at which point a risk might become unacceptable.

Once the risk levels are determined for each sub-elements, the most appropriate method can be chosen in order to manage the main risks (monitoring, periodical inspection, mitigation of consequences, etc.).

5.4 Codes, standards and reliability

Reliability is taking into account in ISO standards as 19903_2014 where the intention is to achieve reliability levels appropriate to the structure. In 1995, design year of NKP design, French (BAEL and BPEL) and Norwegian (NS34E) standards – ref [1] – had been use where Limit State philosophy were driven. Probabilistic approach was not consider anymore and concrete quality, prestressing done, grouting quality and other items impacting were done base on prescriptive approaches. ISO 2394 and ISO 16204 define the serviceability limit states as "a state which corresponds to conditions beyond which specified service requirements for structure or structural element are no longer met". It's completed by *fib* and ISO amendments:

- A definition of the relevant Limit State
- A number of years
- A level of reliability for not passing the Limit State during this period

Unfortunately, a high scatter and variability of achieved construction quality drive the effective reliability of structure, especially for marine structure. A rapid international development of models and procedures for probability-based durability design has taken place in recent years and should be implemented in the next version of codes and standards.

5.5 Inspection plan

Following the RBA, an inspection program is implemented by TOTAL EP Congo aimed to provide a real picture of the unit with constant improvement, i.e. survey results will allow to tune future inspections. Inspection plan tries to identify any defect and the degradation process (chemical attack, corrosion, cracking, coating deterioration, accidents, etc.), define the severity of damage, provide recommendations for repair and provide an image of the condition of the unit to be compared in future campaigns. The main challenge is to draw a correct view of the wall cracks on any place of the unit, not depending of the roper technicians' concrete background.
Cracks maps drawn during the different inspections were compared to anticipated loss of prestressing or design reinforcement. This work was done with a 3D model drawing in order to perform 2 levels of checks:

- Global view of the hull as a beam (hydrodynamics loads)
- Local view of each wall as a separate structure under local loading (water pressure).

6 FLOATING CONCRETE HULL RELIABILITY – NKP CASE

Reliability could be addressed through robustness facing unexpected event, low maintenance level or inspection methods. As far all these parameters could be managed as expected, this kind of structure will provide for owner a very good investment. With 20-year old, NKP concrete hull looks like in perfect shape and soundness and pushes all partners to postpone the end of service of the asset. Cracking of concrete is nearly unavoidable and can be caused by three main factors: material properties, external loads, degradation mechanisms (carbonation, corrosion of embedded steel, etc.). The pre-stress is calculated to counteract friction losses, concrete shrinkage and creep, steel tendons relaxation with time and cable anchorage penetration into the concrete (approximately 30% after 30 years). The prestressing system is not directly accountable as all cables are grouted. TOTAL plans to implement a monitoring system that will become a capital part of the structure as it will be the only way to provide information about the hull that could be used to assess the integrity and the behavior. Reliability of monitoring system must be considered as a key parameter on reliability of the FPU. Existing tools such as optic fiber sensors should address this issue with more confidence and accuracies.
Regarding unexpected event, boat impact or fire and blast that are clearly probable in O&G offshore industry, they are not well known (impact location, area, intensity level, energy, probability, etc…). Definitively, base on NKP feedback, concrete hull provides safe and robust platform.

As an overall durability requirement to the given concrete structure in the given environment, a certain service period could be addressed with a percentage of steel corrosion is reached. NKP built with High Concrete Strength Performance, 54MPa as a minimum compressive strength, and provide a very low porosity material - Ref[2]. It was in the first industrial used of such quality of concrete and it's obvious that the reliability of NKP is linked to this concrete. Typical corrosion issues are done on inserts or steel reinforcements out of cover requirements (50mm), but latest chloride ingress penetration measures after 20 years are less than 20mm (Fig.3)

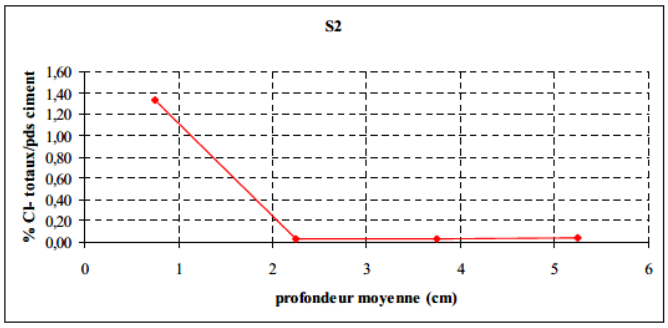

Fig. 3: Chloride Ingress after 20 years

Repairs had been done on deck close to flare due to high temperature not expected during first years. With radiation protection by light roof, no more deterioration should happen.

7 CONCLUSION

Hull integrity of NKP is very simple where no huge maintenance is required. This is probably how reliability of concrete hull is well appreciated by offshore asset management. Postponed the end of the service life, more than 50%, will be the next step to assess the level of reliability. To address this challenge, TOTAL will implement monitoring

system focusing on hull behavior and deduce the level of prestressing. For the next concrete hull, attention will be done on concrete quality through a probability based durability design and process and design of monitoring of prestressing from early age.

ACKNOWLEDGMENTS

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[5] 2006 fib Award for outstanding Concrete Structures, fib bulletins No.36, ISBN 978-2-88394-076-5
[6] Revue TRAVAUX n°779, *Les Travaux d'extension du port de Monaco*, France, Octobre 2000.

8.2 Presentation Kolbjørn Høyland

NTNU Norwegian University of Science and Technology

International Association of Oil & Gas Producers

Experiences with the safety and durability of concrete offshore platforms

Harald A. Rogne / Dr.techn.Olav Olsen AS
Erik B. Holm / Dr.techn.Olav Olsen AS
Kolbjørn Høyland / Dr.techn.Olav Olsen AS

The 3rd Offshore Structural Reliability Conference
OSRC2016
14-16 September, Stavanger, Norway

The following presentation represents the views of Olav Olsen (OO), and may not be fully updated according to the operators' current information

Content

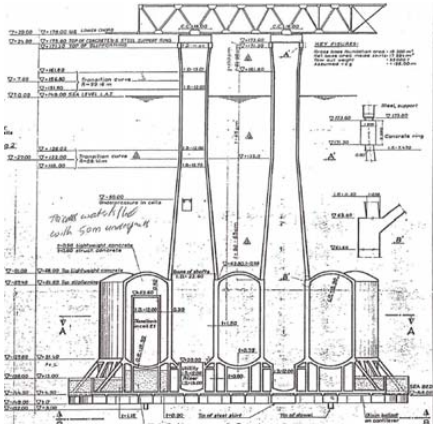
- Introduction/Background
- Governing premises for selection and design of concrete offshore structures
- Lessons learned / Examples
- Challenges and concerns

Introduction and background

- Ekofisk tank installed July 1973
- In 22 years 16 large concrete structures were installed in the Norwegian North Sea.
- 14 fixed and 2 floaters.
- Another 13 in the British, Dutch and Danish North Sea.
- 17 of these built by Norwegian Contractors

Governing premises for selection and design of concrete offshore structures

- Storage
  - Most of the North Sea concrete platforms have oil storage.
  - Upto 2 million bbls storage capacity.
  - Wet storage with underpressure reduce the risk of oil spill.



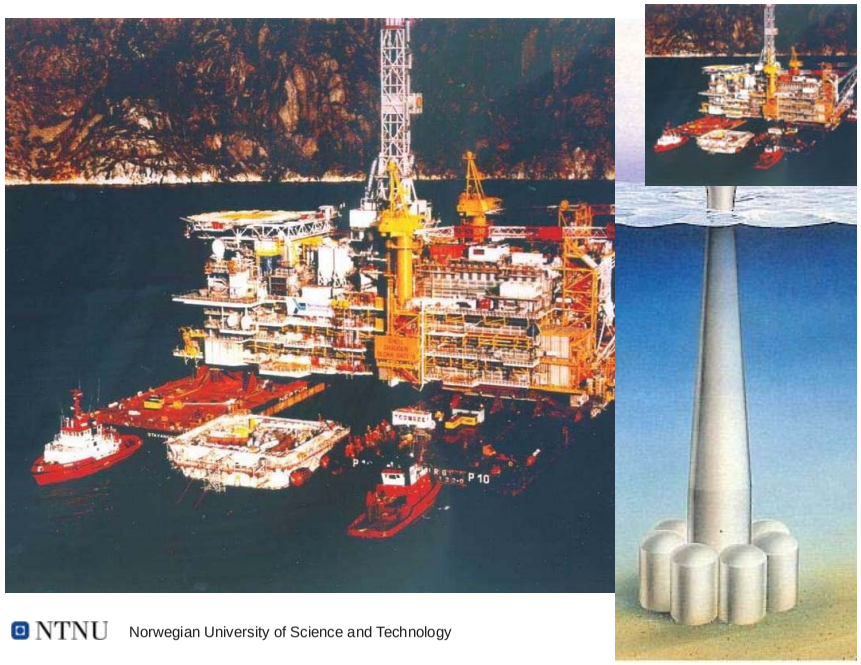
Governing premises for selection and design of concrete offshore structures

- Foundation design
- In extremely soft sea beds, piling can be almost impossible. I such conditions, the large base area of a GBS, combined with penetrating skirts if necessary, will eliminate the need for piling. Only minor or no sea bed preparation is necessary.

Governing premises for selection and design of concrete offshore structures

- Self buoyant and inshore deck mating
  - Heavy topsides can be installed inshore by float over.
  - No high grillage needed.
  - Entire structure is installed offshore by a fleet of tugs.
  - No special purpose heavy lift vessels needed.

Gullfaks C was mated with a deck weighing 47 300 tonnes.



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Governing premises for selection and design of concrete offshore structures

- Local content
  - Concrete structures can be built almost anywhere in the world.
  - Even developing countries have a concrete industry, and skilled concrete workers.

Casting basin for the Sakhalin I and II projects. Outside Vladivostok, Russia.

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Governing premises for selection and design of concrete offshore structures

- Extreme loads
  - Waves
  - Icebergs
  - Sea ice

- Concrete domes and cylinders have an extreme strength against piercing loads like ice loads and ship impact.

Governing premises for selection and design of concrete offshore structures

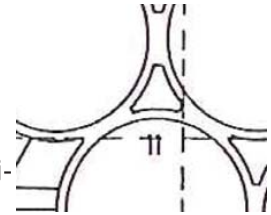
- Long design life
  - Existing structures in the North Sea were typically designed with 30 year design life.
  - Many of them have extended the design life with little or no modification to the substructure.
  - Troll A has a design life of 70 years
  - No concrete structure in the North Sea has been decommissioned due to structural degradation.

Lessons learned

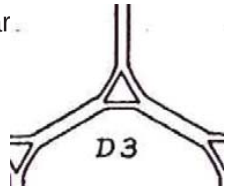
- The lessons learned are gathered through:
  - Incidents during construction
  - Incidents during operation
  - Inspections
  - Damage assessments
  - Re-design due to change in loads or operational procedures
  - Life extensions
  - Removal studies

Incidents during construction

- Beryl A
  - Local tensile cracking (maloperation)
- Statfjord A
  - Crack in cell-joint
  - Lead to improved design of cell-joint and tri-cells



- Sleipner A
  - Loss of substructure due to insufficient shear reinforcement in tri-cell wall
  - The industry have improved engineering procedures as a result of “lesson learned” from Sleipner A



Incidents during operation

- Dropped riserpipe Brent Bravo
  - 36” riser penetrating upper dome
  - 100 m^3/h water leaking into storage cell
  - Repaired by concrete cap
  - Lead to the introduction of protective layer of lightweight concrete on upper domes of later structures.

Incidents during operation

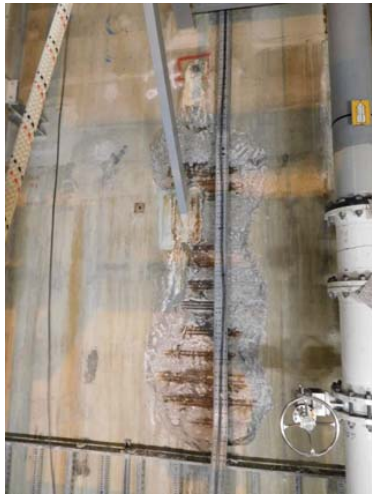
- Ship impact
  - One impact causing damage is reported on the North Sea concrete structures
  - 1.5 m diameter hole in the shaft
  - The shaft was repaired without stopping platform operation.
  - Design criteria for ship impact has been changed to include larger ships.

Damage assessments

- Corrosion of pipes penetrating the concrete walls
  - Pipes fully embedded in concrete are generally well protected
  - Pipes inside the shafts can often be repaired or replaced
  - The challenge is the interface between concrete and air.
  - Based on experience with corroding ballastpipes on the oldest GBSes the newer structures have pipes penetrating concrete made of GRP, high quality steel or even titanium.

Damage assessments

- Gullfaks C – Reinforcement corrosion
  - Concrete reinforcement assumed caused by sea water leakage.
  - No water flow, only moisture
  - 10-20 % reduction in reinforcement in some areas.
  - Structural capacity is assessed based on original design reports. (Ongoing)



Draugen – Post-installation studies

Dated	JOB DESCRIPTION
Nov. 1994	Leakage verification: Skirts and outer lower domes
Nov. 1994	Shear tension selected publications. Modified field theory
1995	Center dome B1
April 1995	Estimation of oil leakage due to temperature increase
July 1995	Draugen contingency planning
Sept. 1995	Increased temperature in storage cells (from 35 to 45)
Des. 1995	Permanent closure of drawdown system in shaft
Des. 1995	Shaft, moment curvature relationship and capacity
March 1996	Permanent closure of skirt pressure system
March 2000	Draugen structure analysis system. SRS
Aug. 2001	Shaft capacity. Design verification.
May 2003	Produced water at 68 deg. Cell integrity.
March 2007	Draugen - Increased Topside Phase II report
2009	Draugen - Increase temperature in shaft
2010/2011	Draugen - Top of shaft modification. Feasibility study

Life extensions / redesign

- Re-design to increase design life normally includes:
  - Updated input
    - Increased wave loads
    - Increased topside weight
    - New ship impact criteria
  - Current design codes
  - Fatigue design
- Extending the operational life of a concrete GBS is in most cases a desktop exercise utilizing:
  - Non-linear analyses
  - Soil data calibrated to actual behavior
  - Concrete ageing (~5 % strength increase)

Troll A - Increased deck weight

Project objective:
Verify that Troll A GBS has sufficient structural capacity when subjected to increased loads from deck, wind and waves

- Installation of two additional gas compressors and other quipment will increase the topside weight with more than 12000 t, up to a total of 44677 t
- New wave loads were determined by extensive model test at Marintek
- Wind loads were increased, due to greater deck surfaces exposure

Troll A - Increased deck weight, stages in strength control

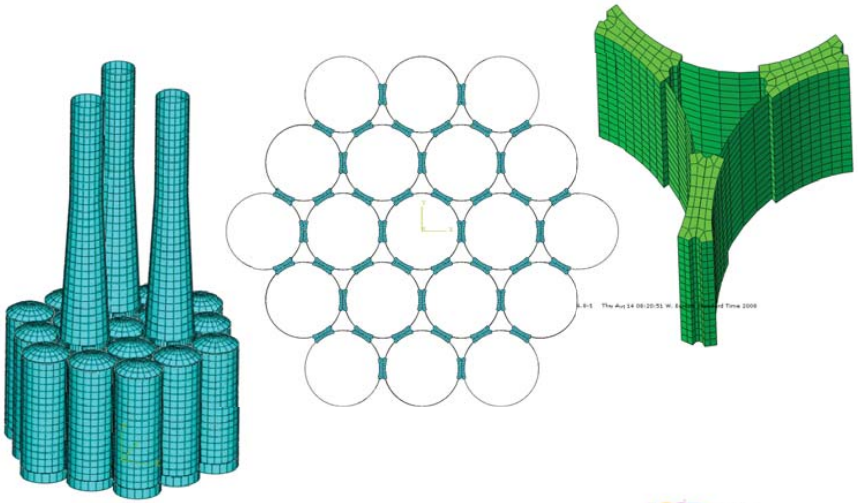
Stage 1	• Reproducing original design results in Multicon
Stage 2a	• Revised environmental loads, 2003-edition of NS 3473, as-built concrete strength, material factor 1,15 for prestressing tendons, tension stiffening factor 0,5 • Implementing as-built reinforcement • Control of crack widths in SLS • Control of max stress in prestressing tendons for response from characteristic wave
Stage 2b	• Increasing max allowable strain in ordinary reinforcement, ε_y, from 2,5 ‰ to max 10‰ when necessary • Accounting for spare capacity in the prestressing tendons
Stage 3	• Considering smoothing and redistribution against neighbouring sections after an assessment of the governing load combination • Local stress field analysis, “strut and tie” etc

Capacity verified

Track-record on documented extended design life and increased topside operating weight

Platform	Design Life	Extended design life	Tow-out topside weight (t)	Max topside design weight (t)	Documented GBS capacity for increased topside weight (t)
Statfjord A	30	+20	19.000	49.000	5.700 (12%)
Statfjord B	30	+20	40.000	61.600	1.400 (2%)
Statfjord C	30	+20	40.000	65.000	6.500 (10%)
Gullfaks A	30	+14	42.500	52.900	12.100 (23%)
Gullfaks C	30	+11	43.000	57.400	11.600 (20%)
Draugen	30	-	20.500	27.800	2.200 (8%)
Troll A	70	-	22.500	32.000	12.600 (39%)

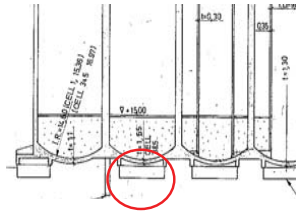
Brent D refloat study, new FEA



Statfjord A removal study, loss of water feed

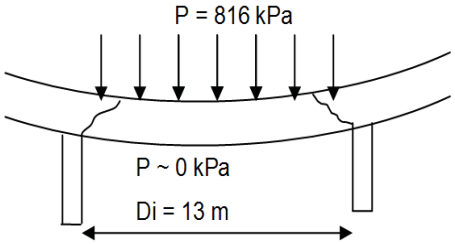
Inside storage cell

- 10.5 meters of solid ballast
- 25 meters of water
- 4.5 barg air pressure
- = 816 kPa



Inside skirt without water feed

- Vacuum ~ 0kPa



Shear force in dome:

$$V = \frac{\frac{\pi D_i^2}{4} \times p \times \gamma}{\pi D_i} = 3445 \ kN/m$$

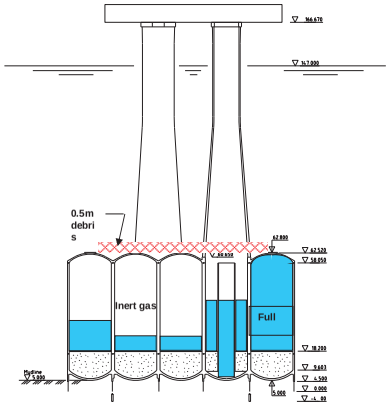
Brent D refloat study, GBS condition

GBS condition

- Leakage in cell 19 due to possible cracks
- Possible damage to concrete in conductor slot #24

Ballasting scenarios

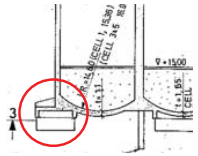
- Uncertainties in cell integrity leads to different ballasting scenarios together with possible removal of drill cuttings and sediments
- High differential pressures are critical for the concrete structure

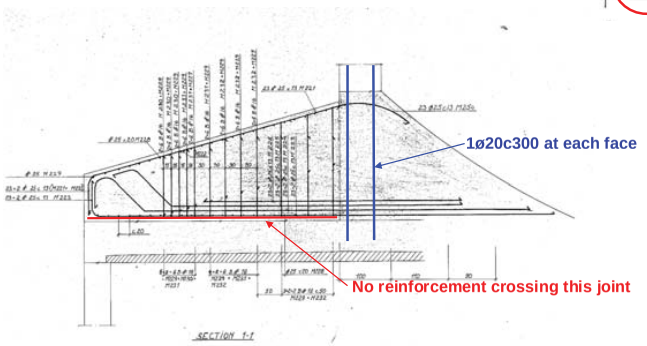


Cell 19 flooded and open to sea (cannot be emptied)
Sediments in cells and drill cuttings in tricells removed

Statfjord A removal study, cantilever

- The cantilever is designed for upwards forces only.
- The beam on top of the cantilever slab is designed after the slab was cast, and is not connected to the slab.
- Analyzed in a Sesam FEM-model





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Our conclusion

- In general the concrete structures are robust.
  - Design life can normally be increased by more than 50% without major repair or modification.
  - Damages caused by accidents can be repaired.
  - Due to the linear design approach used in the original design there is often a margin on the global capacity.
  - The degradation seen so far has been repairable and not critical to the structural integrity.
- Even if the structures are robust, proving capacity for removal has not been possible for the oldest platforms.
- As long as there is available expertise. The structures can be operated for the remaining design life, and most of them can probably have further life extension.

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Keeping the expertise

- Since 1995 no large offshore concrete structures have been constructed in Norway.
- Except the Troll A redesign, none of the studies performed on platforms in operation have the volume needed to transfer knowledge to new engineers.
- With 50-70 years of design life we cannot rely on first hand knowledge from original design. Many of those involved in the design and construction of Condeeps in the seventies are now in or approaching their seventies…

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There is hope…

- Norway is now exporting offshore concrete knowledge to Russia, Italy, Canada and Australia.

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Chapter 9

Session 7: Reassessment of Jacket Platforms in Operation

9.1 Presentation & article by Francis Guede

NTNU Norwegian University of Science and Technology

International Association of Oil & Gas Producers

Risk-Based SIM for Offshore Jacket Platforms

Francis Guédé
Bureau Veritas – Marine & Offshore Division
Neuilly-sur-Seine, France

The 3rd Offshore Structural Reliability Conference
OSRC2016
14-16 September, Stavanger, Norway

Foreword

- API has promoted risk-based approach
- API provides general guidelines for risk-based SIM
- Bureau Veritas method is based on API guidance

Features of the method

- This risk assessment includes
  - Global risk level
  - Local risk levels
- Inspection Strategy includes
  - ⇒ Inspection interval
  - ⇒ General inspection requirements
  - ⇒ Level III survey pre-selected locations (if required)

API Guidance – Risk Categorization

- Exposure Categories

Life Safety Category	Consequence of Failure Category		
	C–1 High	C–2 Medium	C–3 Low
S–1 Manned-Nonevacuated	L–1	L–1	L–1
S–2 Manned-Evacuated	L–1	L–2	L–2
S–3 Unmanned	L–1	L–2	L–3
L – 1: High ; L – 2: Medium ; L – 3: Low			

- Likelihood of Failure Categories

Likelihood of Failure Category	General guidelines
high	RSR < 1 and overload may lead to wave-in-deck load
medium	RSR > 1 but can sustain significant damage after design event
low	Sufficient reserve strength and tolerant to any damage or overload from design event

API Guidance – Inspection Strategy

Risk-Based Inspection Intervals

Risk Category	Inspection Interval Ranges
Higher	3 to 5 years
Medium	6 to 10 years
Lower	11 to 15 years

Default Inspection Program

Interval (Years)	Exposure Category [a]		
	L-3	L-2	L-1
	5–10	5–10	3–5
Level II			
General visual survey	X [b]	X [b]	X [b]
Damage survey	X	X	X
Debris survey	X	X	X
Marine growth survey	X	X	X
Scour survey	X [c]	X [c]	X [c]
Anode survey	X	X	X
Cathodic potential	X	X	X
Riser/J-tubes/caisson	X	X	X
Interval (Years)	**Exposure Category [a]**		
	L-3	**L-2**	**L-1**
	d	**11–15**	**6–10**
Level III			
Visual corrosion survey	X [e]	X [e]	X
Flooded member detection or member close visual inspection	X	X	X
Weld/joint close visual inspection, after cleaning to bright metal	If required	If required	X
Level IV [f]			
Weld/joint NDT	g	g	g
Wall thickness	g	g	g

[a] Exposure category is defined in 5.3.4.
[b] Detection of significant structural damage should form the basis for initiation of a Level III survey in 6.5.1.
[c] If seafloor is conducive (loose sand) or seafloor instability is known/suspected, a scour survey should be performed.
[d] Only required if the results from the Level II survey indicate suspected damage.
[e] Not required if the annual above-water inspection CP survey indicates uninterrupted protection below water.
[f] Only required if the results from the Level III survey indicate suspected damage.
[g] Surveys should be performed as indicated in 5.5.4.3.

Global LoF Assessment

Influencing Factors	
As-Installed Condition	Design Practice
	Structural configuration
	Foundation System
Present Condition	Last Inspection
	Mechanical damage
	Corrosion Protection
	Splash Zone Damage
	Marine Growth
	Scour
	Debris

Influencing Factors	
Platform modification(*)	Deck Load
	Appurtenances Number
Loading exposure	Fatigue Sensitivity
	Appurtenances Exposure
	Damage Susceptibility
	Wave-in-Deck (**)
	Earthquake (**)

(*) not included in LoF score but managed by risk mitigation
(**) treated separately

$$S = \sum_i w_i \cdot S_i$$

S: overall LoF score
S_i: partial LoF score for i−th factor
w_i: weight of the i−th factor

Global LoF – Background

- LoF assessment uses a rule-based scoring approach
- The approach was initially developed by BP AMOCO
  S. DeFranco, et. al. *Development of a RBUI Process for Prioritizing Inspections of Large Numbers of Platforms.* OTC, 1999.
- It has been demonstrated to provide a simple means to developing rational and optimal inspection strategies.
- It has been customized and applied by many other oil & gas companies (e.g. PETRONAS, GUPCO)

Global LoF – Example Scoring Rules

The scoring rules use expert inputs and guidelines from dedicated JIP studies, results published in reference papers.

2 typical examples

- Design Practice Rule
- Robustness Rule

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Design Practice Rule

Design Year	Before 1971 (Pre-RP2A)	1971 – 1979 (Early-RP2A)	After 1979 (Modern-RP2A)
Score	10	6	4
Weighting = 8			

May be further subdivided to introduce additional key API-RP2A dates e.g.

- 1974: joint design criteria
- 1986: 100-year return period design event
- 1993: Revision of environment loading provisions
- 2007: Joint flexibility equations

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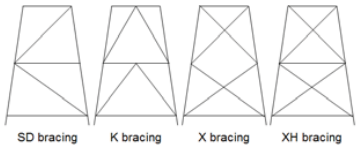
Robustness Rule

Bracing system	Number of legs: 3	4	6	8	8+
K	10	10	8	6	4
SD	10	7	5	4	3
X	6	5	4	3	2
Weighting = 10					



Brace configuration	Number of legs: 3	4	6	8	8+
K	NR	NR	NR	MR	R
SD	NR	NR	MR	R	R
X	NR	MR	R	R	R
XH	MR	R	R	R	R

NR non-robust; MR Moderate robust; R robust

Robustness Matrix (MSL, 2003)

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Global Risk Ranking

- LoF categories with respect to LoF scores' ranges calibrated by defining a set of candidate platforms for each category
- CoF categories are based on operator's specifications
- Default risk matrix is 5x5, unsymmetrical (reflecting risk aversion), with 5 risk levels

Default risk matrix format

Likelihood \ Consequence	1	2	3	4	5
5	IV	IV	IV	V	V
4	III	III	IV	IV	V
3	II	II	III	IV	V
2	I	II	III	III	IV
1	I	I	III	III	IV

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Local Risk Assessment

- Focused on the tubular joints of the jacket structure.
- It uses:
  - results from in-place analysis and fatigue analysis
  - existing local inspection data

Local LoF

- Local LoF uses a rule-based scoring approach

Influencing Factors	
Structural analysis results	Fatigue
	Static strength
Inspection history	Existing inspection
	Reliability of the inspection technique
	Weld crack indication
	Member's damage indication

$$S = \sum_i w_i \cdot S_i$$

S: overall LoF score
S_i: partial LoF score for i−th factor
w_i: weight of the i−th factor

- A larger weight assigned to fatigue
- Inspection history penalizes non-inspected joints/members and joints/members having defects

Local CoF

- Local CoF given by the global consequence of failure reduced by a redundancy factor

$$C_j = C - RF_j$$

C_j: local CoF of joint j
C: global CoF
RF_j: redundancy factor of joint j

- Redundancy factor given in terms of:
  - Number of legs
  - type of member attached to the joint (primary, secondary or tertiary)
  - Punching ratio

Local Risk Ranking

- Relative risk ranking of the tubular joints of the jacket
- Number of joints per risk levels are set out on the risk matrix

Example of local risk levels distribution

Likelihood \ Consequence				
0	0	0	0	0
0	0	0	0	0
0	0	1	0	0
14	85	4	0	0
0	0	0	0	0

Inspection Strategy

- The inspection intervals range from 3 years to 12 years depending on the global risk level

Likelihood \ Consequence				
IV	IV	IV	V	V
III	III	IV	IV	V
II	II	III	IV	V
I	II	III	III	IV
I	I	III	III	IV

Global Risk Level

Likelihood \ Consequence				
5	5	5	3	3
7	7	5	5	3
10	10	7	5	3
12	10	7	7	5
12	12	7	7	5

Risk-Based Inspection Intervals

Inspection Program – General Requirements

Interval (Years)	Exposure Category [a]		
	L-3	L-2	L-1
	5–10		
Level II			
General visual survey	X [b]	X [b]	X [b]
Damage survey	X	X	X
Debris survey			
Marine growth survey			
Scour survey	X [c]	X [c]	X [c]
Anode survey	X	X	X
Cathodic potential	X	X	X
Riser/J-tubes/caisson	X	X	X
Interval (Years)	**Exposure Category [a]**		
	L-3	**L-2**	**L-1**
	d	**11–15**	**6–10**
Level III			
Visual corrosion survey	X *	X *	X
Flooded member detection or member close visual inspection	X	X	X
Weld/joint close visual inspection, after cleaning to bright metal	If required	If required	X
Level IV [f]			
Weld/joint NDT	g	g	g
Wall thickness	g	g	g

[a] Exposure category is defined in 5.3.4.
[b] Detection of significant structural damage should form the basis for initiation of a Level III survey in 6.5.1.
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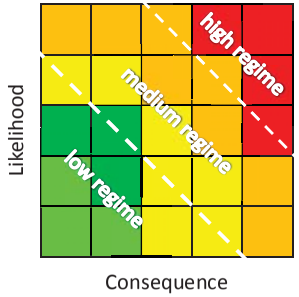


Level III (CVI/FMD) inspections scope

- A weighted average model provides risks scores to rank joints in order of priority for inspection
- Number of joints to be inspected given by the mean risk score
- Two options to define level III scope of work:
  - CVI on preselected tubular joints,
  - or FMD on the members attached to the preselected joints.

Application

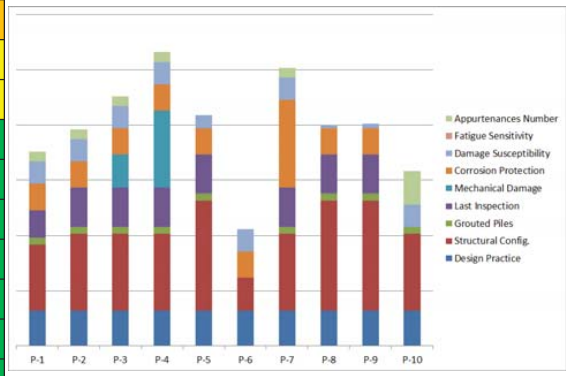
Platform ID	Design year	Nb. legs	bracing	function	manned?
P-1	1993	8	K	production	no
P-2	1995	4	SD	production	no
P-3	1993	4	SD	wellhead	no
P-4	1998	4	SD	wellhead	no
P-5	1993	4	K	quarters	yes
P-6	2004	8	X	production	no
P-7	1993	4	SD	wellhead	no
P-8	2004	3	SD	support	no
P-9	1995	3	SD	flare	no
P-10	2002	4	SD	wellhead	no

Global Risk results

Sample platforms' risk data

Platform ID	LoF	CoF	Risk level
P-4	5	2	IV
P-3	4	2	III
P-7	4	2	III
P-1	3	2	II
P-2	3	2	II
P-5	3	2	II
P-10	3	2	II
P-6	2	2	II
P-8	3	1	II
P-9	3	1	II

Sample platforms' Likelihood of failure scores



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Local Risk Results – Platform P-4

Distribution of local risk levels of P-4

0	6	0	0	0
7	38	0	0	0
87	20	0	0	0
0	0	0	0	0
0	0	0	0	0

Local risk levels on P-4 3D model

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Conclusion

- The method includes a global risk assessment and local risks assessment.
- An acknowledge rule-based scoring approach is used to assess likelihood of failure
- The global risk provides inspection intervals and general inspection requirement based on API guidance and inspection trends.
- Local risks assessments serve to perform relative risk ranking of the structural components and to define level III inspection scope if required.

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Risk-Based Structural Integrity Management for Offshore Jacket Platforms

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Risk-Based Structural Integrity Management for Offshore Jacket Platforms

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ABSTRACT: This paper presents a method for risk assessment and inspection plan development as part of the risk-based structural integrity management of offshore jacket platforms. The method provides a global risk assessment for the whole platform's structure and local risk assessment for the platform's structural components (e.g. tubular joints). The global risk assessment uses semi-quantitative or quantitative approaches depending on the available platform's data. The semi-quantitative method is used either at a high level to perform relative risk ranking of platforms in a fleet in order to identify the platforms most at risk and which require more inspection focus or a detailed risk analysis; or at the unit level to define inspection interval and general inspection requirements. The quantitative method involves either structural analysis results (e.g. reserve strength ratio) with a dedicated metocean hazard curve or structural reliability method; and it is used at the unit level only to define inspection interval and inspection requirements. The local risk assessment uses also semi-quantitative or quantitative approach. The semi-quantitative method is used to provide local risk ranking of the structural components of a platform, which allows, if required, local inspections' scope to be defined. The quantitative method involves a probabilistic fatigue method to define inspection plans for selected tubular joints subjected to fatigue and the failures of which are critical for the overall platform structure. The inspection strategy and program, developed by the method presented in this paper, are focused on the routine underwater inspections and are based on the recommended practice of the American Petroleum Institute for the structural integrity management of fixed offshore platforms. An example of application of the method is set out by showing the results of a project carried out by Bureau Veritas.

KEY WORDS: structural integrity management; risk assessment; offshore jacket platform.

1 INTRODUCTION

The American Petroleum Institute (API) has released in December 2014 its first standard [1] for the structural integrity management (SIM) of fixed offshore platforms. This standard emphasizes the value of using risk-based approach to develop effective inspection strategy and program and provides guidelines to develop risk-based inspection strategies. However, only general guidelines are given for the risk assessment and for the preselection of survey locations. In particular, descriptive criteria are defined to assign a risk level to a platform. Moreover, the factors, to be considered in selecting survey locations to provide representative overall structural condition, are listed, but no method is proposed to select those locations.

Bureau Veritas contributed to the joint industry project for the development of the API standard for SIM [1] and it has implemented a risk-based SIM for offshore jacket platforms that incorporates the API requirements. The purpose of this paper is, especially, to present the method set up by Bureau Veritas for risk assessment and for developing risk-based inspection strategy including inspection intervals, general inspection requirements and scope of the local inspections (e.g. inspection of welded joints) if required.

The risk assessment, in this paper, includes the platform's global risk level and the local risk levels of the platform's structural components (e.g. tubular joints). The global risk assessment allows risk-based inspection intervals and general inspection requirements to be defined. It proposes a gradual risk evaluation in terms of available platform's data from a semi-quantitative assessment based on key platform's data (e.g. robustness, last inspection findings, manning status, and functionality) to a quantitative assessment based on structural analysis results or involving structural reliability method. The local risk assessment performs local risk ranking of the tubular joint of the jacket structure to identify local inspections scope if required. A semi-quantitative assessment is proposed to select the tubular joints or members where close visual survey (CVI) or alternatively flooded member detection survey (FMD) should be applied. A fatigue-based probabilistic method is also implemented to determine optimal inspection plans for selected tubular joints, the fatigue failure of which is critical for the overall structural integrity, and which joints require crack monitoring by non-destructive technique (NDT).

This paper sets out, first, an overview of the API guidance on risk-based SIM for offshore jacket structures. Then, the proposed method is presented. This method is also illustrated on some platforms that were involved in a SIM project carried out by Bureau Veritas.

2 OVERVIEW OF API GUIDANCE FOR RISK-BASED SIM OF JACKET PLATFORMS

The API-RP-2SIM [1] includes guidance for risk-based approach to SIM of offshore jacket platforms. It provides general guidelines for assigning a risk category to the platforms in terms of the exposure category and the likelihood of failure. The exposure category is defined with respect to life safety exposure and consequence of failure including the environmental and the economic impact (Table 1). A description of the relevant factors to consider for determining the life safety exposure category and the level of consequence of failure is also given. The standard allows qualitative, semi-quantitative, or fully quantitative methods to be used in assessing the level of likelihood of failure. However no detail is given on how to implement those methods. Only general guidelines are defined for the assessment of likelihood of failure category.

Table 1. Exposure Matrix (API-RP-2SIM)

	Consequence of failure category		
Life safety category	C-1: high	C-2: medium	C-3: low
S-1: manned non-evacuated	L-1	L-1	L-1
S-2: manned evacuated	L-1	L-2	L-2
S-3: unmanned	L-1	L-2	L-3

L-1: high; L-2: medium; L-3: low

The risk-based inspection strategy is specifically concerned with the routine underwater inspections. However, it requires that a baseline inspection was conducted and it should use the findings from the above-water inspections and the eventual post-event inspections. The API gives detailed recommendations for determining inspection strategy from the risk categorization, including risk-based inspection intervals and work scope, survey techniques and deployment methods. Typical ranges of risk-based inspection intervals (Table 2) are provided with respect to the platform risk level along with a description of the inspection scope of work. The associated risk-based inspection program has to be a minimum level II survey (i.e. general underwater visual inspection and corrosion protection survey), according to the API classification of survey levels, but has to specify if higher survey levels e.g. level III (i.e. CVI or FMD) and level IV (i.e. NDT) are required.

Table 2. API risk-based inspection intervals.

Risk category	Inspection interval
higher	3 to 5 years
medium	6 to 10 years
lower	11 to 15 years

When risk-based approach is not adopted, API provides a default inspection program based on the exposure category only (Table 3).

Table 3. API default inspection program.

	API exposure category		
	L-3	**L-2**	**L-1**
Level II survey			
GVI	X(1)	X(1)	X(1)
Damage	X	X	X
Scour	X(2)	X(2)	X(2)
Debris	X	X	X
Marine growth	X	X	X
Cathodic potential	X	X	X
Anode	X	X	X
Riser/J-tube/Caisson	X	X	X
Level III survey			
Visual corrosion	X(3)	X(3)	X
CVI or FMD	X(4)	X	X
Weld/joint CVI			X
Level IV survey(5)			
Weld/joint NDT	X	X	X
Wall thickness	X	X	X

(1) detection of significant damage should be the basis for initiation of level III survey
(2) should be performed if seafloor is conducive or if seafloor instability is known or suspected
(3) not required if the annual above-water CP survey indicates uninterrupted protection below water
(4) required only if results from the level II survey indicate suspected damage
(5) required only if results from the level III survey indicate suspected damage

3 GLOBAL RISK ASSESSMENT

3.1 Overview

The global risk assessment includes a global Likelihood of Failure (LoF) and a global Consequence of Failure (CoF) assessment. The global consequence, including life safety, environmental and financial consequences, is assessed by one of the following methods:

- a qualitative method using descriptive criteria,
- a semi-quantitative method using a scoring process.

The global likelihood is assessed by one of the following methods:

- a semi-quantitative method using a rule-based scoring approach,
- a method which uses the available structural analysis results with a dedicated metocean hazard curve,
- a structural reliability method.

3.2 Global CoF

The guidelines provided by the API are used to develop qualitative and semi-quantitative assessment methods for the CoF. Those methods are based on the variables that have been highlighted by the API as affecting the CoF. The qualitative method uses descriptive criteria in terms of the listed variables

while the semi-quantitative method uses a scoring process that assigns a consequence score to a platform in terms of the listed variables.

3.2.1 Life-safety consequence

Life safety consequence depends on two main variables:

- whether personnel are likely to be exposed when an undesirable design event occurs
- the evacuation capability for platforms that are usually occupied.

The number of exposed personnel on one hand and the degree of difficulty of the evacuation on the other hand are considered to provide various levels, respectively to the first and second variables. The degree of difficulty of the evacuation depends on:

- the distances involved
- the number of personnel to be evacuated
- the capacity and operating limitations of the evacuating equipment
- the type and size of docking/landings, refueling, egress facilities on the platform
- the environmental conditions anticipated to occur throughout the evacuation effort.

3.2.2 Environmental consequence

The environmental consequence depends on two main variables:

- whether structural failure, loss of mechanical integrity or directly applied loads can cause rupture of equipment containing hydrocarbon liquid or scour gas e.g. topside vessels, risers, pipeline, conductors, etc.
- the impact on the environment of hydrocarbon liquid or scour gas released.

The first variable is directly linked to the expected amount of hydrocarbon liquid or scour gas released and will depend for example on the storage and processing capability of the platform. The second variable is linked to the proximity of the platform to the shoreline or to environmentally sensitive areas such as coral reefs, estuaries, and wildlife refuges.

3.2.3 Financial consequence

The financial consequence depends on two main variables:

- the importance of the structure to the owner's overall operation
- the level of economic losses if a structural failure occurs.

In particular, the importance of a platform depends mainly on its functionality but other parameters could be included by the operator; and the economic losses depend on the size of the platform and whether the platform failure could damage an adjacent platform or infrastructure.

3.3 Global LoF

3.3.1 Semi-quantitative method

The method uses a rule-based scoring process. This approach was initially developed by BP Amoco for its own fleet and presented at the OTC conference in 1999 [2]. It is based on a similar approach being developed by the API for refineries and chemical plants [3]. It has then been customized and applied by other oil & gas companies (e.g. PETRONAS, GUPCO) and some studies have been published (e.g. [2], [4] and [5]).

Rules are defined in the form of tables that guide the user through the process of assigning scores to relevant factors (e.g. structural characteristics, present condition, etc.) which influence the platform's susceptibility to failure. A weight is also assigned to each influencing factor with respect to how strongly it affects the overall LoF of a platform. The overall LoF score is given by the weighted sum of the factors' scores.

$$S = \sum_i w_i \cdot S_i \tag{1}$$

where S_i is the i-th factor's score and w_i is its weighting.

Then, LoF categories are defined with respect to ranges of the overall score. Those ranges are calibrated on a representative set of platforms.

a) Factors influencing the global LoF

The factors that affect the failure susceptibility of an offshore jacket platform can be divided into four broad categories:

- As-installed condition
  - Design practice, including year of design or year of installation
  - Structural configuration, including number of legs and bracing system
  - Foundation system, including type of foundation (e.g. mudmat, pile system) and whether the piles are grouted or not

- Present condition
  - Last inspection, including year and level of last inspection
  - Damaged members
  - Missing or cut members
  - Corroded members or remaining wall thickness
  - Flooded members
  - Corrosion Protection (CP) system, including potential readings and/or anode depletion
  - Splash zone damage and/or corrosion
  - Marine growth
  - Scour
  - Debris

- Platform modification
  - Topside weight change
  - Appurtenances (e.g. risers, conductors, caisson) number change

- Loading exposure
  - Wave-in-deck
  - Appurtenances (e.g. risers, conductors, caisson) exposure
  - Fatigue sensitivity

– Earthquake

Especially for the assessment of the present condition factors, due consideration should be given to the full history of inspections, repairs and structural assessments to better judged the present condition of the platform and any deviation from last inspection findings should be confirmed by the client.

In the method of this paper, some factors are excluded or treated separately. The platform modification factors (e.g. topside weight change, appurtenances number change) are not taken into account in the likelihood scoring process, since they are not managed within inspection strategy but rather by risk reduction actions. Thus, when the perceived platform's change is deemed critical, a fitness-for-purpose assessment should be performed; and risk reduction measure should be undertaken if the structure is not fit-for-purpose.

The wave-in-deck and earthquake loading exposure are so critical for the LoF that they should be treated separately. Thus, when these loads apply to a structure, its fitness-for-purpose should be assessed. A risk reduction measure should be undertaken if the structure is not fit-for-purpose; otherwise how they downgrade the baseline LoF should be determined and applied.

b) Scoring rules for the global LoF

Simple qualitative rules have been developed for the relevant influencing factors. For example, the design practice factor accounts for the improvement over the years of the definition of metocean design loads and of structural analysis process, which tends to increase the platforms' strength. 3 eras have been identified in the evolution of the design practices (Figure 1), and denoted Pre-RP2A, Early-RP2A and Modern-RP2A [6]. A design practice rule has been developed from this classification (Table 4), and has been used in many SIM projects (e.g. [2], [4] and [5]).

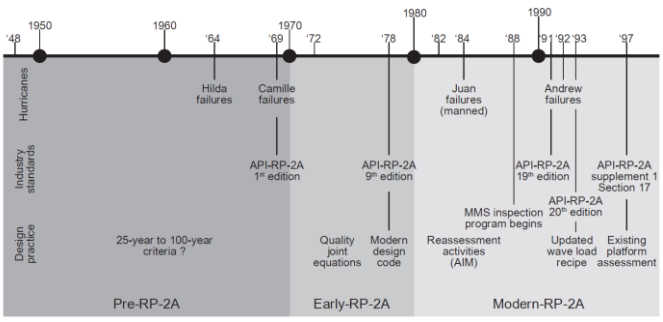

Figure 1. Evolution of Design Practices.

Table 4. Design Practice Rule.

Design Year	Pre 1971 (Pre-RP2A)	1971 – 1979 (Early-RP2A)	After 1979 (Modern-RP2A)
Score	10	6	4

This common rule may be further subdivided, if required, to introduce additional key dates of the design practice evolution, e.g.:

- introduction of joint design criteria in 1974
- 100-year return period extreme design event defined explicitly in 1986
- substantial revision to the environment loading provisions in 1993
- introduction of joint flexibility equations in 2007, in particular guidelines on how to include them in structural analyses are provided in API design standard [7].

A common rule for structural configuration (Table 5), with respect to number of legs and bracing system, has been also used in many SIM projects. This rule reflects that robust structures are more damage tolerant and have a lower likelihood of collapse failure. It is based on the robustness matrix (Figure 2) provided by the Joint Industrial Project (JIP) on significance of damage to fixed offshore platforms [8].
The scoring rules related to platform's condition and loading exposure use guidelines from dedicated JIP studies as well as results published in research papers (e.g. [9], [10], [11], [12]). Specific inputs to the operators as well as specific inputs to the platform's location may be accounted for in defining these rules.

Table 5. Structural Configuration LoF Scoring Rule.

Number of legs	3	4	6	8	8+
Bracing K	10	10	8	6	4
Bracing SD	10	7	5	4	3
Bracing X	6	5	4	3	2

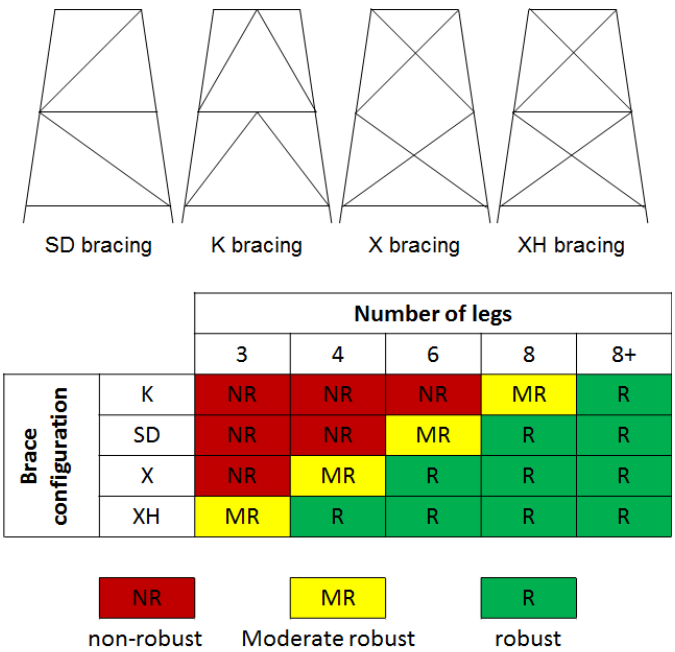

Figure 2. Robustness Matrix (JIP MSL, 2003).

c) Categorization of global LoF

Five categories for the LoF are considered, namely: Very high, High, Medium, Low and Very low category. The ranges, in which their respective scores lie, are calibrated by defining for each category a set of candidate platforms. However, it is difficult to rely on statistics of structural failure data because

historical data of platform's structural failure are sparse. It is also difficult to rely on statistics of platform's structural capacity data, which would require too many structural analysis computations to get a representative set of data. In practice, the calibration database includes platforms data, the LoF levels of which are assumed based on experience and expert judgment. Yet, uncertainty may affect this type of assessment since different people may have different opinions for the same problem, even though they are expert. In this case, some classical methods can be used to reduce or minimize the uncertainties, such as taking the average of the experts' respective estimates, or trying to reach a consensus estimate during a workshop meeting gathering many experts. Moreover, LoF levels may be assumed based on an operator's requirements or risk perception, which may be different from one operator to another. In this case, guidelines should be provided by the SIM analyst to the operator under consideration, in order to ensure that its specific estimates comply with minimum standard requirements or local regulation requirements.

Two ways to calibrate LoF categories have been encountered in the studies that were performed and presented in the literature. In the initial approach [2], LoF categorization is based on the assumption that platforms designed according to modern structural detailing practice to resist present day design environmental loads have the lowest LoF. Then, the factors that affect the original strength, the maximum design loads and the degradation of the strength are used to benchmark any individual platform's LoF against this "ideal platform". Another approach is to define the LoF categories from the statistical distribution of the LoF scores of the platforms in a given fleet having a large number of platforms. This allows only relative likelihood assessment, specific to the fleet under consideration, to be developed. In this case, the categories limits are given by fractiles of the cumulative density function of the platforms' LoF scores. Example are provided by the 5%, 50%, 70% and 95% fractiles for 5 LoF categories, assuming the 5% of the platforms have higher LoF and lower LoF.

In this paper, both existing calibration approaches are used depending on the purpose of the risk assessment. When the assessment is carried out at a high level in order to provide relative risk ranking of platforms in a fleet, then the LoF categories are based on the statistical distribution of the LoF scores of the platforms in the fleet. When the assessment is carried out at a unit level to develop an inspection strategy, then assumptions are made on the LoF level of a set of platforms arbitrarily defined in each one of the LoF categories. In this case, relevant assumptions are made according to:

- general guidelines from the API standard [1]
- available knowledge on inspection trends and sensitivity of platform's capacity to its structural characteristics and to its condition, which are provided in research results from JIP report and reference papers
- inputs from the operator to take its risk perception into account.

3.3.2 Quantitative Method Based on Structural Analysis Results

This method uses typically the reserve strength ratio (RSR) to compute the LoF by means of a dedicated metocean hazard curve. The RSR is provided by an ultimate strength analysis (i.e. pushover analysis). However, structural analysis for a jacket structure does not necessarily go up to the ultimate strength analysis especially when the maximum value of the punching ratio (UC) provided by a design level structural assessment is lower than 1. In this case, provided that the assessment is carried out in compliance with the current API design requirements, the RSR is assumed to vary between 1.8 and 2.5 with respect to the platform robustness [13]. This assumption is used to device a RSR value and to deduce a LoF level.

a) Metocean hazard curve

Figure 3 shows a typical metocean hazard curve. It includes a portion where the structure is subjected to wave-in-jacket only and a portion including wave-in-deck load after air gap is reached by the wave crest. It is usually specific to the platform location since the metocean conditions depend on the water depth. When the curve includes wave-in-deck load, it is also specific to the platform itself because air gap is specific to each platform. Therefore, a dedicated metocean hazard curve should usually be developed for each platform structure.

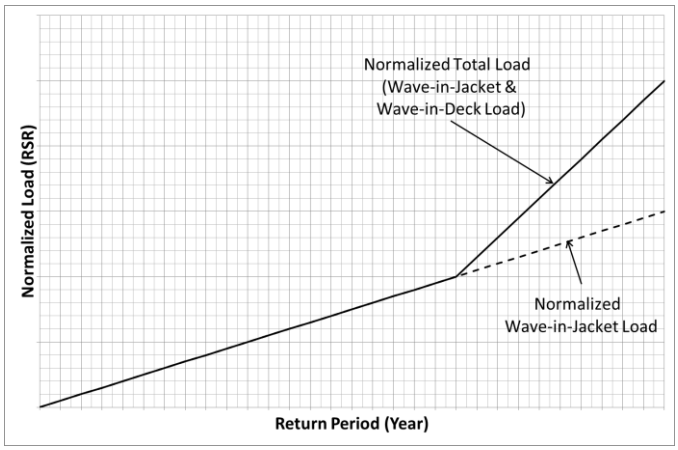

Figure 3: Typical platform specific metocean hazard curve.

Metocean hazard curves are developed by so-called response based approach. It requires a structural response model, usually approximated by an empirical formula in terms of the metocean parameters e.g. wave height, current speed, etc. The structural responses considered for offshore jacket structures are the base shear load and the overturning moment, however, base shear is the most used structural response because ultimate strength from pushover analysis are usually expressed in terms of maximum base shear load that the structure can withstand. This approach allows structural reliability to be computed from the statistical distribution of the structural response. The distribution of the structural

response is obtained from metocean data using the response model. Two main approaches exist in the literature to develop metocean hazard curve:

- a simplified method described for example by Energo [14]
- a rigorous method proposed by Tromans [15] based on storms statistics.

b) Simplified method

The base shear model is assumed to follow a simple empirical formula:

$$BS = C \cdot H_{\max}{}^{\alpha} \tag{2}$$

where BS stands for the base shear load, C and α are parameters of the model and $H_{\max}$ is the annual maximum wave height. The parameter C is not of interest since it disappears by normalization of the load by the 100-year return period load. The parameter α varies between 1.2 and 2.2 from experience in terms of the platform location, but it is common to set α to 2. Figure 4 shows typical metocean hazard curves with respect to the parameter α when $H_{\max}$ is assumed to follow a Rayleigh distribution.

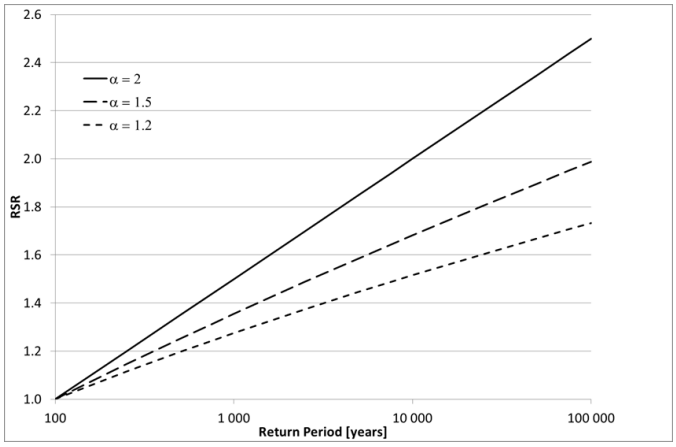

Figure 4: Typical metocean hazard curves in terms of power parameter α of the base shear model and assuming $H_{\max}$ follows a Rayleigh distribution.

A detailed load model can be considered to account for current speed and possible wave-in-deck loading [14]. It reads:

$$BS = \left[C_1 + C_4 \cdot \left(H_{\max} - H_d\right)\right] \cdot \left[H_{\max} + C_2 \cdot u\right]^{C_3} \tag{3}$$

where

- H_d is the smallest wave height with a crest that will reach the bottom of the cellar deck
- u is the current speed
- C_1, C_2, C_3 and C_4 are the parameters of the model reflecting the following:
  - C_1 is a general parameter for the overall platform shape (e.g. number of legs 4 or 8)
  - C_2 is the parameter for the current
  - C_3 is similar to the power parameter α
  - C_4 is the parameter for wave-in-deck loading

To determine the parameters C_i, a series of increasing wave heights are run past a 3D computer model of the platform under consideration and the load model is fitted to the results.

Let us show, for illustration purpose, hazard curves derived from a standard wave scatter diagram. The standard wave scatter diagram is provided by the recommendation 34 of International Association of Classification Societies (IACS) [16] and describes wave data of the North Atlantic derived from Global Wave Statistics. Although this scatter diagram is not to be applied to fixed offshore structures but to ships, it is used here for illustration purpose only.

The cumulative distribution of the wave height in a given sea state is assumed to follow a Rayleigh distribution, which is a reasonably accurate model:

$$F_H(h) = P(H \le h) = 1 - \exp\left[-2 \cdot \left(\frac{h}{H_S}\right)^2\right] \tag{4}$$

where H_S is the significant wave height of the sea state under consideration.

Let us denote n_w the average number of waves in the sea state and n_S the average number of times the sea state occurs in one year.

$$n_w = \frac{t}{T_Z} \tag{5}$$

where t is the time duration of a sea state in seconds (sea state time duration is usually set to 3 hours) and T_Z is the zero-crossing period of the sea state.

$$n_S = p \cdot \frac{T}{t} \tag{6}$$

where p is the probability of occurrence of the sea state under consideration derived from the scatter diagram; and T is the duration in seconds of one year.

The cumulative distribution of the annual maximum wave height is given by the following formula under the assumption of independence of the waves, which is a conservative assumption.

$$F_{H_{\max}}(h) = \prod_{i,j} \left(F_{H,i,j}(h)\right)^{n_{S,i,j} \cdot n_{w,i,j}} \tag{7}$$

where the pair i,j represents the pair $\left(H_{S,i}, T_{Z,j}\right)$ of the scatter diagram.

Thus, if $L(h)$ denotes the load model, the cumulative distribution of the annual maximum load is obtained by:

$$F_{L_{\max}}(l) = F_{H_{\max}}(h), \quad \text{with } l = L(h) \tag{8}$$

Finally, the metocean hazard curve is obtained by inverting the following equation:

$$1 - F_{L_{\max}}(l) = \frac{1}{RP} \tag{9}$$

where RP is the return period.
Figure 5 shows hazard curves obtained from the standard scatter diagram and a simplified load model for base shear given by: $BS \propto h^{\alpha}$.

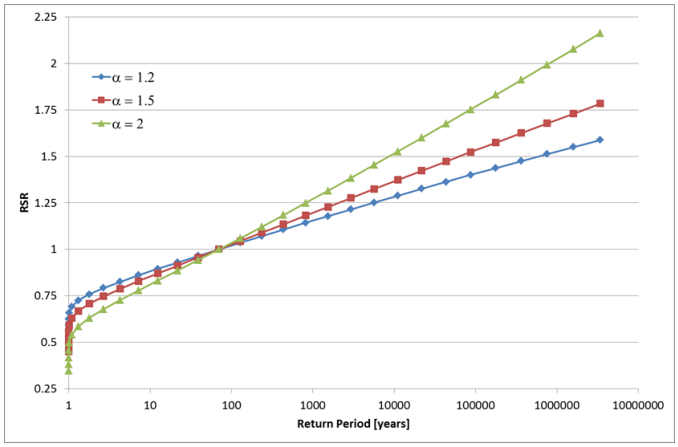

Figure 5: Example hazard curves from scatter diagram for North Atlantic (IACS Rec. 34 [14]).

c) Rigorous method by Tromans [15]
The improvement brought by Tromans' method is twofold:

- The method uses a structural response model defined in terms of most of the metocean environmental parameters, which allows joint metocean conditions to be generated.
- The method is based on storm statistics rather than sea states, which allows the correlation of the successive occurrences of the sea states to be taken into account.

Thus, this method allows the conservatism of the results to be significantly reduced in comparison to the simplified method.

A storm is defined by the most probable extreme structural response within that storm rather than the maximum structural response to the individual waves. In fact, the most probable extreme is function of several of the larger structural responses; therefore, this storm definition should be less sensitive to noise [17].

The method uses the asymptotic properties of extremes to provide the long term distribution of the structural response (e.g. base shear load) that allows the metocean hazard curve to be developed.
The load model is based on the stick models for statically responding, drag dominated structures. It is expressed for the base shear for example by:

$$BS = A_1 u^2 + A_2 u a T_Z \phi \cos\theta_c + A_3 \phi^2 a^2 + A_4 \phi a^2 \frac{\cos\theta_c}{T_Z} + A_5 \phi^2 \frac{a^3}{T_Z^{\,2}} + A_6 \phi^2 a^2 T_Z^{\,2} + A_7 W^2 \cos\theta_w \tag{10}$$

where:

- a is the linear crest elevation
- ϕ is the directional spreading factor
- T_Z is the zero-crossing period
- u is the depth integrated current
- W is the one minute sustained wind speed
- θ_c is the angle between mean wave and current direction
- θ_w is the angle between mean wave and wind direction

Over the duration of a sea state, all the metocean parameters are treated as constants with the exception of crest elevation which describes individual waves. The model's parameters A_1 to A_7 depend on the configuration of the structure and the attack direction. They are obtained by using a classical least square method to fit the load model to base shear numerical values obtained by running analysis on a structure consisting of a one diameter column roughened from mud-line to sea level and smooth above. A representative set of input variables are selected to carry out the structural analyses. The model has been demonstrated to provide accurate estimation of the base shear load e.g. [15] and [17].

From the theory of extreme value distribution, the distribution of extreme individual crest elevation converges to an asymptotic form conditional on the most probable extreme value:

$$P\left(a_{\max} \middle| a_{mp}\right) \propto \exp\left[-\exp\left[-\ln N \cdot \left(\left(\frac{a_{\max}}{a_{mp}}\right)^2 - 1\right)\right]\right] \tag{11}$$

where,

- a_{mp} is the most probable extreme value of the crest elevation a,
- $N = T_s / T_Z$ with T_s being the time duration of the storm and T_Z being a wave period i.e. the zero-crossing period.

The wave drag force contributes the most to the base shear when extreme waves are considered. Since wave drag force component is proportional to a^2, the shape of the distribution of the extreme base shear load is assumed to be given by the distribution of a^2. Thus, by replacing a^2 by BS in equation (11) the asymptotic model of the distribution of the extreme base shear within a storm is expressed as follows:

$$P\left(BS_{\max} \middle| BS_{mp}\right) \propto \exp\left[-\exp\left[-\ln N \cdot \left(\frac{BS_{\max}}{BS_{mp}} - 1\right)\right]\right] \tag{12}$$

where,

- $BS_{\max}$ is the maximum value of the base shear wave loads,
- BS_{mp} is the most probable value of base shear wave loads.

Time series of metocean parameters are required to identify storms especially from significant wave height H_S time series. Thus, since larger H_S are more of interest to predict extreme conditions, a threshold value is introduced to select larger values of H_S and break them into storms. An optimum threshold can be selected using Mean Excess Plot. The Mean Excess Plot represents the mean excess against possible thresholds values, where the mean excess is defined as the average of the difference between a given threshold and the H_S that are larger than it. This Mean Excess Plot approach has been used in the paper of Li et al. [18] to get suitable threshold to define storms. In this approach, the most linear region of the Mean Excess Plot provides a range where a suitable threshold can be selected. Following that process, sample storms histories are identified for each direction sector.

Then the most probable base shear BS_{mp} can be calculated for each storm. Indeed, from the theoretical distribution of the extreme base shear in Eq. (12), $P\left(BS_{\max}\middle|BS_{mp}\right)=1/e$ when $BS_{\max}=BS_{mp}$, where $P\left(BS_{\max}\middle|BS_{mp}\right)=1/e$ is the empirical distribution of the extreme base shear within the storm *s*. Solving this equation for each storm, a sample set of values of BS_{mp} is obtained. Then, the long-term distribution of BS_{mp} can be estimated by fitting a typical extreme value distribution model (e.g. GPD, Weibull, Gumbel, etc.) to the set of BS_{mp} data.

Finally, the distribution of the extreme base shear is obtained by convolution of the distribution of the extreme base shear within a storm and the distribution of most probable base shear within that storm as follows:

$$F_{BS}(x)=\int F_{BS|BS_{mp}}(x)\cdot f_{BS_{mp}}(x_{mp})\cdot dx_{mp} \tag{13}$$

Assuming independent storms such that storm arrivals can be treated as a Poisson process, the probability distribution of the extreme base shear on the time duration *Q* is:

$$F_{BS|Q}(x)=F_{BS}(x)^{\vartheta Q}\approx\exp\left[-\vartheta Q\cdot\left(1-F_{BS}(x)\right)\right] \tag{14}$$

where ϑ is the mean arrival rate of storms. Then, the annual distribution of the maximum base shear is now given by:

$$F_{BS|1\,\text{year}}(x)\approx\exp\left[-\vartheta\cdot\left(1-F_{BS}(x)\right)\right] \tag{15}$$

From this distribution, the metocean hazard curve can be drawn.
It is commonly assumed that the long-term distribution of maximum base shear follows an exponential form:

$$F(BS_{\max})=1-\exp\left(-\frac{BS_{\max}}{\beta}\right) \tag{16}$$

where β is the distribution parameter. In this case, hazard curves are shown to depend on the occurrence rate of storms only. Indeed, for a mean occurrence rate of storms ϑ, the distribution of annual maximum base shear reads then:

$$F(BS_{\max})=\exp\left[-\vartheta\exp\left(-\frac{BS_{\max}}{\beta}\right)\right] \tag{17}$$

Thus the base shear BS_{RP} with a return period of *RP* is given by $1-F\left(BS_{\max}\right)=1/RP$, which yields:

$$BS_{RP}=\beta\cdot\left(\ln\vartheta+\ln RP\right) \tag{18}$$

Defining hazard curve by normalizing to the 100 – year return period value we get:

$$\frac{BS_{RP}}{BS_{100}}=1+\frac{\log(0.01RP)}{\log(100\vartheta)} \tag{19}$$

3.3.3 Quantitative method using structural reliability

The objective of the structural reliability method is to compute explicitly the probability of collapse failure of a platform, including uncertainties in:

- the gravity force (dead and variable actions)
- the wave-in-jacket force
- the wave-in-deck force
- and the capacity of the components participating in the collapse mechanism (i.e., uncertainties due to fabrication imperfections, material strength and soil capacity).

The probability computation may also include uncertainties in the modeling of those variables.

The structural reliability method is one of the structural assessment methods which API recommends in the alternative assessment methods.

Structural reliability method is normally performed as part of a new design and in the assessment of existing structures, but it can be used in decision analysis to support inspection strategies and programs. In the latter case, the computed probability is combined with the consequence of failure to derive the risk level from which the inspection strategy is to be defined.

Direct computation of the probability of failure is possible only in simple cases e.g. two random variables involved. In the general case (i.e. more than two random variables involved), it requires many structural analyses to be run.

The probability of failure is usually computed by using response surface approach [19]. Typically, applying response surface approach in the case of offshore jacket structures will consist in approximating the load (e.g. the base shear load) with an empirical function in terms of the random variables

that describe the sources of loading (e.g. wave height, wind speed, deck load and eventually wave-in-deck load). This empirical load function can be given by the load models that are commonly used to develop metocean hazard curves and that are given by the equations (2), (3) and (10) for the simplified method, the simplified method including wave in deck load and the Tromans' method respectively. The empirical formula can also be given by a multidimensional polynomial (e.g. quadratic polynomial model) which is classically used in implementing response surface approach.

Classical least square method is used to fit the load function model to a number of sampling points from the failure surface function. The sampling points are given by a combination of possible values of the input random variables. The selection of the sampling points is based on experimental design techniques.

The limit state function used to compute the collapse failure probability is simply given by the ultimate load (e.g. the ultimate base shear load), which stands for the resistance, minus the extreme environmental load. As stated above, the extreme environmental load is approximated by an empirical load function given for example by Eqs. (2), (3) or (10). This empirical load function is denoted by:

$$L(WIJ, WID, SF|\theta) \tag{20}$$

where,

- *WIJ* represents extreme environmental load on the jacket,
- *WID* represents extreme environmental load on the deck,
- *SF* represents static forces i.e. permanent and variable actions (dead and live loads) and wind forces,
- θ represents the direction of the environmental load.

Then, using the definition of the Reserve Strength Ratio (RSR) as:

$$RSR = \frac{L_u}{L_d} \tag{21}$$

where,

- L_d is the design load,
- L_u is the ultimate load,

the limit state function is finally expressed as follows:

$$g = RSR \cdot L_d - L(WIJ, WID, SF|\theta) \tag{22}$$

Note that a failure surface is required for each environmental loading directional sector. Then the total return period for platform collapse should be:

$$\frac{1}{RP_{total}} = \sum_{i=1}^{ndir} \frac{1}{RP_i} \tag{23}$$

where, RP_i is the return period to collapse in the ith direction and RP_{total} is the total return period for platform collapse.
Separate failure surfaces are also required for structural and foundation failure modes.

Finally, the probabilities of failure or equivalently the return periods are computed using Monte Carlo simulation or First Order Reliability Method (FORM) or Second Order reliability Method (SORM) [20].

Let us show a simple example of reliability analysis that involved, in addition to the extreme environmental load on the jacket, uncertainty in the structural capacity of the jacket. The random structural capacity is represented by a random ultimate capacity (i.e. RSR) which is assumed to follow a log-normal distribution,

$$f_R(r) = \frac{1}{r \cdot \sigma \cdot \sqrt{\pi}} \exp\left[-\frac{(\ln r - \mu)^2}{\sigma^2}\right], \text{for } r > 0 \tag{24}$$

where μ and σ are respectively the mean and the standard deviation of $\ln r$. The best estimate of RSR by an ultimate strength analysis is assumed to be the median of that distribution and the analysis will investigate different values of its coefficient of variation.

The failure surface reads:

$$g(RSR, L) = L_d \cdot RSR - L \tag{25}$$

where L and L_d are respectively the empirical load function and the design load.

The probability of collapse failure is then simply given by the following probability integral:

$$P_f = \text{Prob}(g \le 0) = \int (1 - F_L(L_d \cdot r)) \cdot f_R(r) \cdot dr \tag{26}$$

The wave load is defined by the simple load model (i.e. Eq. (2)) with a power α set to 2, i.e. $L \propto (H_{\max})^2$, and the metocean wave climate is described by the standard scatter diagram provided by IACS recommendation 34 [16]. Figure 6 shows the corresponding hazard curves including uncertainty in the ultimate strength given by various coefficients of variation of the RSR log-normal distribution.

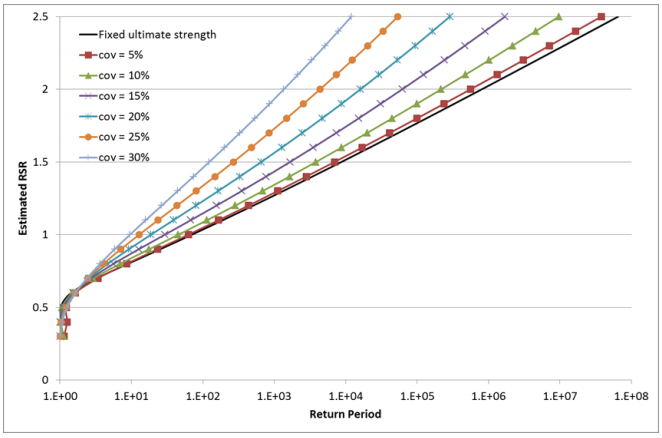

Figure 6: Examples metocean hazard curves including uncertainty in the ultimate strength.

3.4 Global Risk ranking

The global risk level is assessed using a dedicated risk matrix in terms of the global likelihood and consequence of failure. Risk matrices are generally operators specific. By default, the current method considers 5 risk level categories with an unsymmetrical format to reflect risk aversion (Figure 7).

4 LOCAL RISK ASSESSMENT

4.1 Semi-quantitative method

Details on this assessment method are set out in a Bureau Veritas methodological document [21]. An overview is given in the sequel.

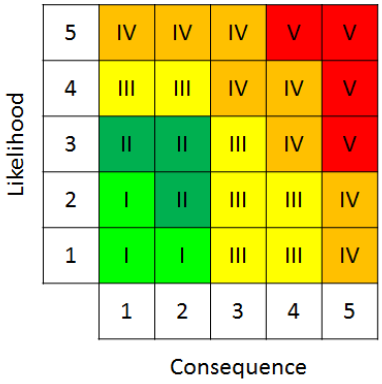

Figure 7. Default risk matrix format.

4.1.1 Local LoF

The local LoF assessment uses a rule-based scoring approach. It is given by the weighted sum of partial scores assigned to factors that influence the LoF of the joint under consideration. The factors that affect local LoF are divided into two main categories:

- Structural analysis local results:
  - fatigue damage
  - static strength
- Inspection history:
  - existing local inspection
  - inspection indication if inspected
  - reliability of the inspection technique if inspected

Simple scoring rules have been developed for each influencing factor. Structural analysis results allow the likelihood of failure to be estimated as a function of stress and fatigue damage, while inspection history penalizes this likelihood to account for observed defects on inspected joints and members or uncertainty on the condition of non-inspected joints and members.

The fatigue scoring rule depends on the fatigue damage provided by the fatigue analysis. A larger weight is assigned to fatigue which is considered to be the most important driver of the local failure assuming that all the punching ratios are lower than 1.

The static strength score depends on the punching ratio provided by the in-place analysis. This factor is critical for local failure only when an exceptional overloading occurs; therefore a lower weight is assigned to this factor. It serves mainly to compare the LoF of joints that have approximately the same fatigue damage.

The existing inspection score penalizes joints which have not been inspected previously to account for the uncertainty on their current condition.

For the joints which have been inspected previously, the inspection indication score penalizes joints for which defect was found either on the welded joint itself or on the members attached to it. The score for the reliability of the inspection technique penalizes less accurate techniques (e.g. NDT is assumed more accurate than CVI or FMD).

Like the global LoF, five categories are considered for the local LoF. The ranges in which the scores lie are calibrated on a set of representative joints data, the failure susceptibilities of which are assessed by engineering judgment.

4.1.2 Local CoF

The local consequence is given by the global consequence of failure reduced by a redundancy factor:

$$CoF_i = CoF - RF_i \tag{27}$$

where CoF is the global consequence of failure and RF_i the redundancy factor. Thus, the local consequence of failure of a non-redundant structural component is almost equal to the global consequence, while it is significantly reduced for a redundant component.

The redundancy factor is given in terms of:

- the number of legs of the platform
- the type of member attached to the joint (e.g. primary, secondary or tertiary member)
- the punching ratio of the joint to account for the possible stress redistribution after the failure of the joint.

4.1.3 Local risk ranking

The local risk ranking provides only relative risk ranking of the platform tubular welded joints. It uses, by default, the same unsymmetrical risk matrix format as the global risk ranking (Figure 7). The numbers of joints per risk level category are set out on the matrix to show the distribution of the local risk levels.

4.2 Quantitative method

The quantitative method involves a full probabilistic approach and allows an inspection plan for an individual welded joint subjected to fatigue to be developed [22]. It is applied to some selected tubular joints which are reported to have higher risk of fatigue failure and the failures of which are critical to the structural integrity of the overall jacket structure.

The computation of the probability of fatigue failure is based on a crack growth model in two dimensions given by Paris

law. Indeed, the method intends to develop inspection plans for crack monitoring. However, the parameters of the crack growth model are usually unknown, especially the distribution parameters (e.g. mean, standard deviation) of the random variables. In practice, those unknown parameters are calibrated so that the crack growth model meets the fatigue performance provided by a relevant S-N curve.

The optimal inspection plan is given by the one that minimizes the expected operational cost, including inspection and maintenance cost and failure cost.

For a joint, the failure of which is so critical for the integrity of the platform structure that it is no more compliant with its approved structural performance criteria, a maximum acceptable probability of fatigue failure must be specified and used as a constraint in finding its optimal inspection plan. This maximum acceptable probability of fatigue failure is computed as follows:

$$P_{fatigue}^{\max} = \frac{P_{collapse}^{\max}}{P_{collapse|fatigue}} \tag{28}$$

where,

- $P_{collapse}^{\max}$ is the maximum acceptable annual probability of collapse failure of the platform, and it is directly deduced from the structural performance criteria e.g. standard requirement [1] for manned platforms is to withstand the 2500 – year metocean load which corresponds to $P_{collapse}^{\max} = 4 \cdot 10^{-4}$;
- $P_{collapse|fatigue}$ is the annual probability of collapse failure of the platform in damage condition, assuming the fatigue failure of the joint under consideration; it is obtained by identifying on the platform metocean hazard curve the return period corresponding to the reserve strength ratio of the platform structure in this damage condition.

The method can be time consuming if many joints are selected especially for larger platforms. In this case, a generic approach, proposed by Straub and Faber in [23], is used to speed up the process in finding an optimal inspection plan. The generic approach develops ahead a database containing suitable inspection plans for a set a set of so-called generic representations of the typical joints, which are defined in terms of so-called generic parameters e.g. detail type, thickness, fatigue damage, etc. Then, when inspection plans for the individual joints in the structure under consideration are to be determined, they are obtained from that database through an interpolation procedure.

5 INSPECTION STRATEGY AND PROGRAM

5.1 Risk-based inspection intervals

The API-RP-2SIM provides guidelines for the risk-based inspection intervals with respect to three risk levels (Table 2). In the current method, the inspection intervals range from 3 years to 12 years with respect to the global risk level (Figure 8). However, the inspection interval may be adjusted to account for the design life, the present condition of the CP system or operational feasibility and regulations.

5.2 Inspection scope of work

In accordance with API recommendations, the inspection program should be a minimum of level II survey and damage or deterioration found during a level II survey is the basis to trigger a Level III or Level IV inspection.

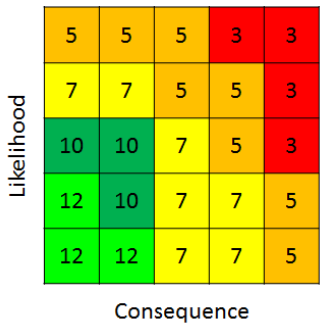

Figure 8. Inspection intervals.

The inspection scope of work is based on the default inspection program provided by the API. The method considers the respective inspection programs per exposure category (Table 3) as three inspection regimes, denoted low regime for exposure category L-3, medium regime for exposure category L-2 and high regime for exposure category L-1. Those inspection regimes are applied with respect to the risk level as indicated on the Figure 9.

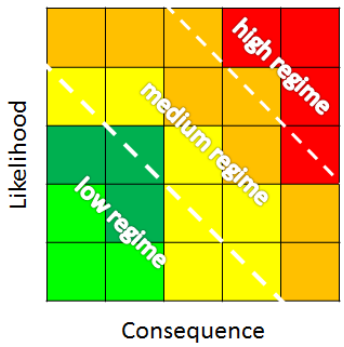

Figure 9. Inspection program.

When level III surveys are required, they should include pre-selected joints or members with respect to the local risk ranking, in addition to the locations where damage are suspected from level II survey.

A weighted average model is used to provide risks scores to the tubular joints to rank them in order of priority for inspection. This model involves the local likelihood and consequence of failure along with the local risk level and gives more weight to the consequence of failure to reflect risk aversion. Then, the mean value of the risks scores is used to

determine the percentage of joints with the higher risks scores to be inspected.

The method proposes two options to define the local inspection scope of work, either:

- applying CVI on preselected tubular joints,
- or using FMD technique on the members attached to the preselected joints.

6 APPLICATION

An application example of the current method is given by showing some results from a previous SIM project performed by Bureau Veritas. A set of 10 platforms is considered. Table 6 sets out their main characteristic data e.g. design year, number of legs and bracing configuration,…

Table 6. Sample platforms' data.

Platform ID	Design year	Nb. legs	bracing	function	manned ?
P-1	1993	8	K	Production	no
P-2	1995	4	SD	Production	no
P-3	1993	4	SD	Wellhead	no
P-4	1998	4	SD	Wellhead	no
P-5	1993	4	K	Quarters	yes
P-6	2004	8	X	Production	no
P-7	1993	4	SD	Wellhead	no
P-8	2004	3	SD	Support	no
P-9	1995	3	SD	Flare	no
P-10	2002	4	SD	Wellhead	no

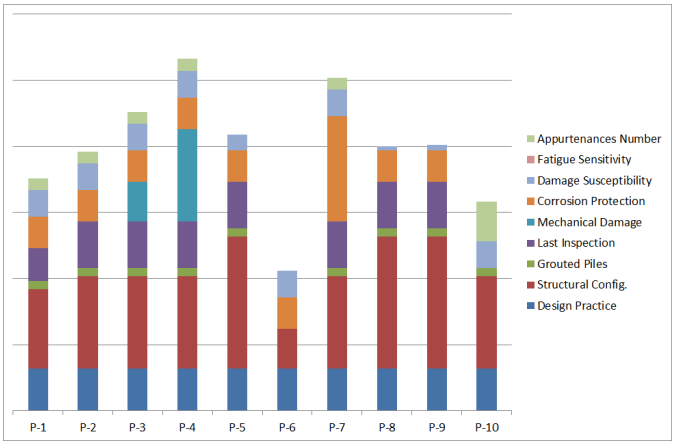

Figure 10. Likelihood of failure scores.

Figure 10 shows the likelihood of failure scores of the platforms with the contribution of the individual influencing factors. They are all modern-RP2A designed platforms, most of them having the same structural configuration. Their risk levels are therefore differentiated by their present condition (from last inspection results), functionality and manning level (Table 7).

Table 7. Sample platforms' risk data.

Platform ID	LoF	CoF	Risk level
P-4	5	2	IV
P-3	4	2	III
P-7	4	2	III
P-1	3	2	II
P-2	3	2	II
P-5	3	2	II
P-10	3	2	II
P-6	2	2	II
P-8	3	1	II
P-9	3	1	II

For illustration purpose, the local risk ranking results for the platform P-4 are shown. The distribution of the risk levels on the dedicated risk matrix is set out on Figure 11, as well as on the corresponding tubular joint on a 3D model from Bureau Veritas SIM software on Figure 12.

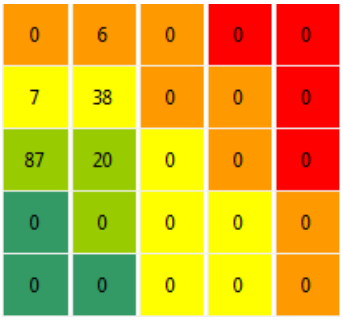

Figure 11. Distribution of local risk levels of P-4 platform.

7 CONCLUSION

A method used by Bureau Veritas for risk-based SIM of offshore jacket platforms has been presented in this paper. It has been shown to provide risk-based inspection strategies and programs in compliance with the first standard for SIM released by the API in December 2014.

The risk assessment method comprises semi-quantitative and quantitative assessment levels, which can be selected in terms of the platform data that are available and the required level of accuracy of the assessment. The scoring approach used for semi-quantitative risk assessment is simple and includes all the drivers affecting the failure susceptibility of a platform. Thus, in addition to providing the risk level, it provides also an understanding of that risk. Concerning the quantitative methods, they implement existing approaches for computing probability of failure. In particular, a simple application of structural reliability method is shown using a simple load model in terms of directional scatter diagrams, which are usually an available data, and a lognormal distribution assumed for the structural resistance (i.e the Reserve Strength Ratio).

The method presented in this paper has been effectively implemented on an industrial project as shown by the results provided for illustration.

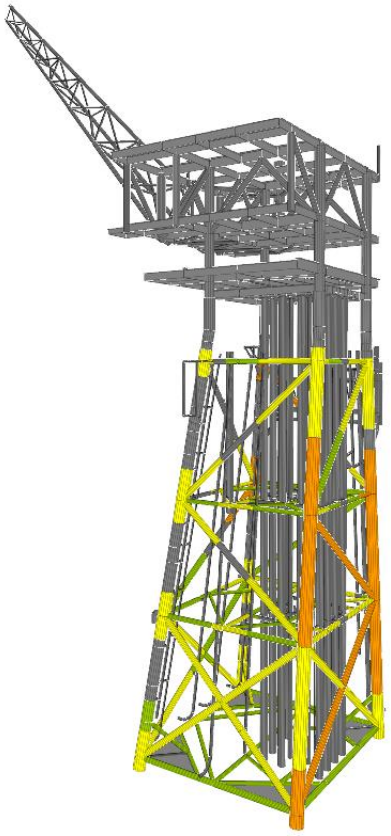

Figure 12. Local risk levels on P-4 platform's 3D model.

ACRONYMS

AIM	Asset Integrity Management
API	American Petroleum Institute
CP	Cathodic Protection
CoF	Consequence of Failure
CVI	Close Visual Inspection
FMD	Flooded Member Detection
FORM	First Order Reliability Method
GVI	General Visual Inspection
IACS	International Association of Classification Societies
LoF	Likelihood of Failure
NDT	Non Destructive Testing
RSR	Reserve Strength Ratio
SIM	Structural Integrity Management
SORM	Second Order Reliability Method
UC	Unity Check

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9.2 Presentation by Thomas Langford

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Geotechnical issues for assessments

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Victor Smith, NGI
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The 3rd Offshore Structural Reliability Conference
OSRC2016
14-16 September, Stavanger, Norway

Geotechnical issues for assessments

- Background
- Overview of challenges and opportunties within assessments
- General principles
- Piled foundations
- Anchors for floating systems

Background

- Foundation and anchoring systems for (floating) structures are frequently assessed for a variety of reasons
- Ageing offshore developments and changes in rules/codes increase the need for assessments and related geotechnical studies
- Important to have a clear and consistent approach which is understood by all stakeholders: engineers, owners and certifying authorities
- A successful analysis can make the difference between retaining existing structure/components or large investment costs for replacement

Some challenges

- Often limited geotechnical data (few borings) for original site and structure
- Finding geotechnical data in the project archives → luckily we like dusty reports
- Quality of data often lower than current practice: lower quality samples, lab testing or in-situ test data
- 'Bespoke' design methods may require specific test data

Typical geotechnical engineer?

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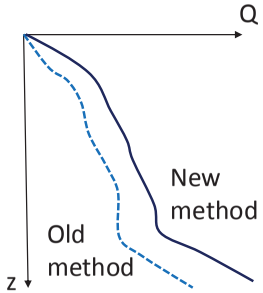
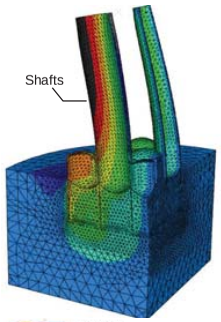
Some opportunities

- New data may be available for the site or area, for instance from new structures
- Improved design methods usually increase reliability and may demonstrate better performance than the original design
- Geotechnical capacity and stiffness of pile and anchors changes with time
- Installation data gives useful input, e.g. layering, interface remoulding etc.
- Monitoring data sometimes available, e.g. for Condeep GBS platforms





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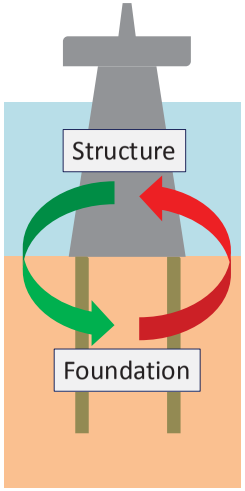
General principles

- Assessment shall be 'up-to-date' and reflect 'best-expected' performance
- Geotechnical performance depends on applied loads (and vice versa)
- Structural-geotechnical interface is critical to reliable analysis. Work as a team!
- Often unknown whether stiffer or softer performance governs. Best to perform analyses without additional factors
- Start with comprehensive review of available data, both before and after original installation



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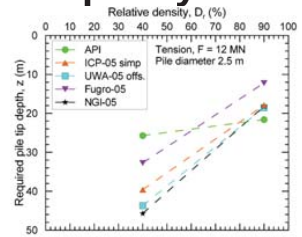
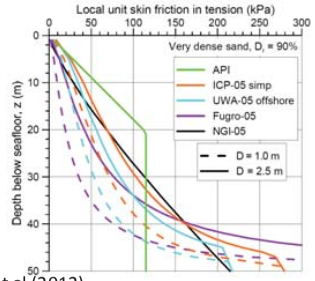
Piled foundations – reference capacity

- Piled foundations designed according to empirical methods
- Limited field test data available when original design methods were created
- Earlier design methods less reliable compared to newer pile load tests
- Several 'new' CPT-based sand methods and also clay methods recently developed
- Reliability of new methods improved (ref. Ongoing JIP, presentation from Farrokh Nadim)
- For North Sea sites with dense sand layers: capacity is often greater than estimated by API-method





Ref: Knudsen et al (2012)

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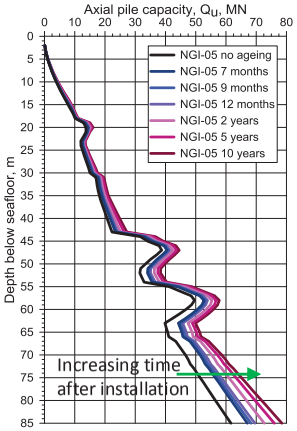
Piled foundations - ageing

- Many things get better with age… including piled foundations
- Axial capacity of piles is driven by properties at pile soil interface
- Field tests show that interface properties improve with time due to set-up and 'ageing'
- Several good references (e.g. NGI JIP)

Wine…..

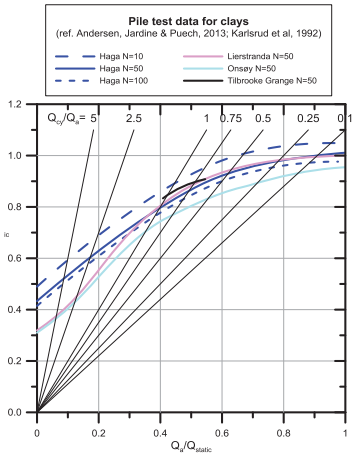
Cheese…..

Even concrete!



Piled foundations – cyclic response

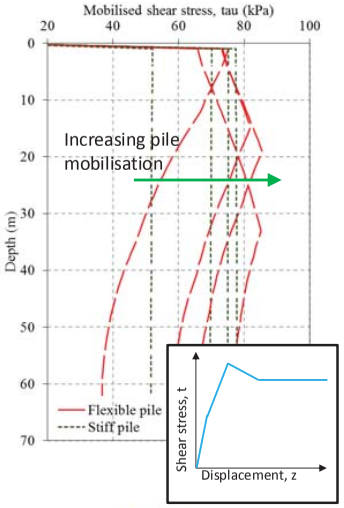
- Cyclic loading influences capacity and stiffness
- Soil degradation depends on material properties and load history
- Viscous rate effects in clay provide counter-balancing benefit
- 2-way cyclic may give significantly lower capacity
- 1-way cyclic loading has less impact



Ref: Andersen (2015)

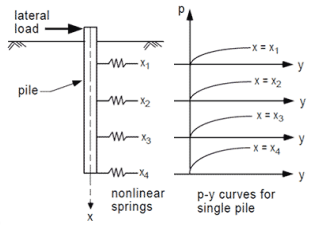
Piled foundations – effect of displacement

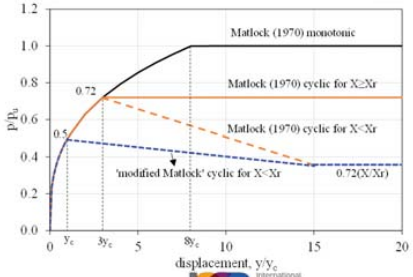
- Load displacement behaviour shall be accounted for
- Ultimate reference capacity does not include this
- Shaft resistance along pile varies with mobilisation
- Post-peak 't-z' response reduces overall skin friction and thus ultimate pile capacity, especially for long piles in clay
- Jacket calculations should include the 'displacement-compatible capacity'



Piled foundations – lateral response

- Lateral pile response important for assessment of critical structural element
- Pile-soil response provided in terms of p-y curves
- Earlier p-y curves calibrated for limited field data & smaller piles
- Recent work has provided improvements, especially for piles in (softer) clay
- P-y curves shall be realistic to capture critical structural response

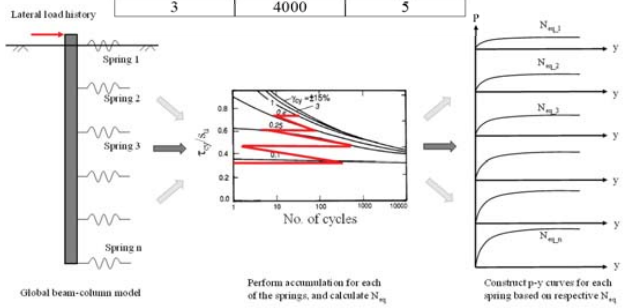


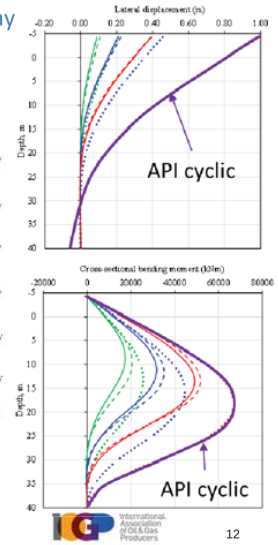


Piled foundations – lateral response

'NGI Pile' cyclic p-y beam-spring model 84'' x 40 m pile in clay

Parcel No.	Load, kN	No. of cycles
1	2000	20
2	3000	10
3	4000	5



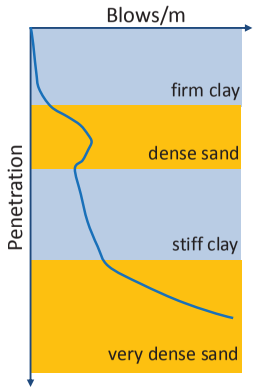


Cyclic loading procedure combines rate effects and fatigue

Ref: Zhang et al (2016)

Piled foundations – soil properties/layering

- Soil properties and layering should be re-evaluated for assessments
- Layering can be assessed from driving records. North Sea piles driven into dense sand may be confirmed from driving records.
- Soil properties (e.g. sand density) can be inferred from driving data, but need 'ground truthing' with regional data including CPTUs and blowcounts
- Variation in blowcounts across site should also be compared with variation in CPTU data – do they match up?



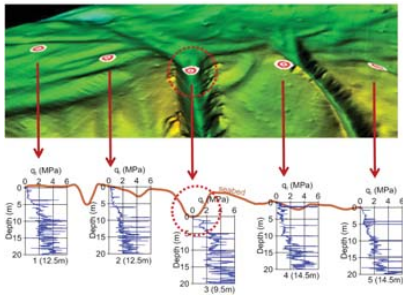
Suction anchors - general

- Suction anchors used often on deepwater projects. Relatively 'new' (post 1995)
- Floaters have different design issues than fixed structures
- Different design methodology than other anchor types (piles, drag anchors)
  - Less empiricism → more reliable
  - Well described in codes e.g. DNV-RP-E303
  - Direct benefit of good SI and lab data
- Mainly used in clay (also sand)
- Due to more recent vintage, usually more and better geotechnical data available

Ref: Langford et al (2012)

Suction anchors – use of installation data

- Designed tolerances (i.e. uncertainties) for penetration, misorientation and tilt
- These uncertainties can be 'eliminated' once anchor is installed
- Installation data can be used to understand variation in properties at actual anchor location
- Local variations important for 'shallower' foundations; e.g. effects of iceberg ploughmarks on Haltenbanken



Ref: Langford et al (2012)

Njord FSU - overview

- Floating storage unit with mooring system installed in 1997
- 4.9 m diameter, 8 – 10 m penetration
- Soft to firm clay over stiff boulder clay
- Lifetime extension project; mooring system will be retained
- Anchor design verified by NGI for updated (partially increased) loads
- Important to include realistic assessment of soil properties and capacity

Anchor FSU 7

Ref: Shin et al (2016)

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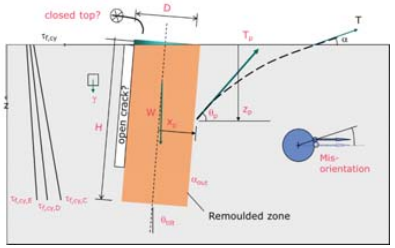
Njord FSU – first check

- 'First-pass' check
  - Original design profiles
  - 'As-built' tilt and misorientation better than design tolerances
  - No allowance for cyclic loading
  - Standard '2D' analysis model
- 3 anchors didn't pass first check
  - FSU 5 (increased load)
  - FSU 6 (short penetration, increased load)
  - FSU 8 (short penetration)
- Additional action required to document sufficient capacity!



Anchor location	Length (m)	Penetra-tion (m)	Padeye load (kN)	Padeye angle (°)	Material factor	Failure mode
FSU1	9.00	6.85	3780	17.5	1.5	/pos.
FSU2	10.10	9.90	2003	34.9	1.53	\neg.
FSU3	10.10	9.90	1935	43.1	1.52	\neg.
FSU4	10.10	9.90	4803	21.2	1.50	\neg.
FSU5	10.10	9.90	7933	17.5	1.10	\neg.
FSU6	10.10	8.30	7974	16.3	0.94	\neg.
FSU7	10.10	9.90	6240	23.4	1.47	\neg.
FSU8	9.00	7.30	5545	15.9	1.20	\neg.

Ref: Shin et al (2016)

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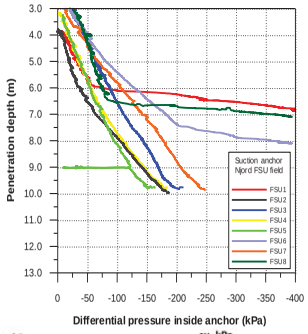
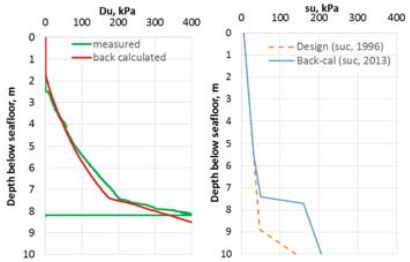
Njord FSU – soil profiles

- Soil profiles re-evalauted based on back-analysis of installation data (suction)
- Iteration of parameters:
  - Sensitivity
  - N_{kt} factor
  - Anistropy factor
  - Cone resistance
- Appropriate and reasonable adjustments to layering and strength profiles





Ref: Shin et al (2016)

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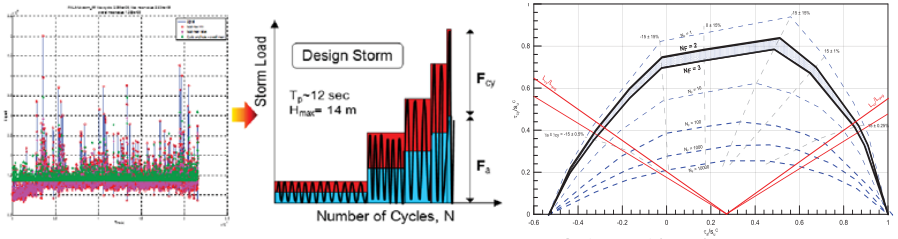
Njord FSU – analysis model

- Cyclic soil behaviour - 'fatigue' (–ve) and viscous rate effects (+ve)
- Requires load history (in this case provided by MARINTEK)
- Additional capacity of ~20% compared to static reference strength



Ref: Shin et al (2016)

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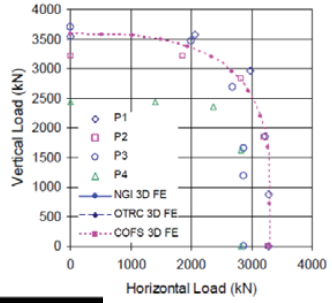
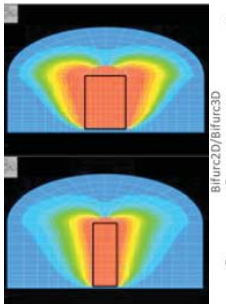
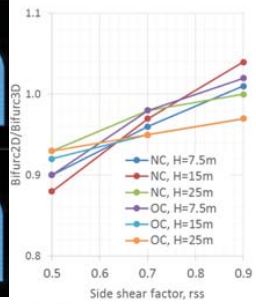
Njord FSU - analysis

- Simplified 2D models often used for suction caisson analyses. Approaches and results vary across the industry
- Appropriate calibration required to provide more reliable results and optimised design
- 3D calibration of results for Njord allowed increase in 'side shear' factors and around 15% additional capacity







Ref: Jostad and Andersen (2015)

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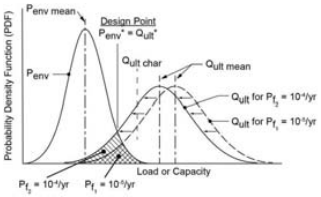
Njord FSU – summary

- Significant increases in holding capacity obtained through appropriate incremental refinements in assessment process
- All suction anchors in the mooring system retained for the lifetime extension
- Not necessarily representative for all anchors – original design had significant margin
- Next stop – Njord FPU!
- Reliability-based evaluation of safety factors and capacity (similar to approach presented by Farrokh for piles)



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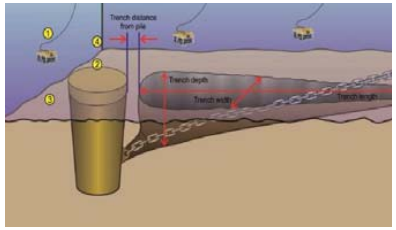
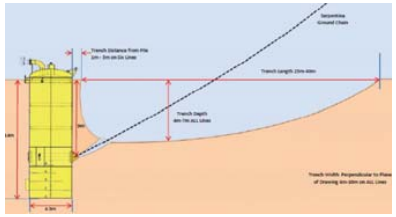
Suction anchors - trenching

- Reported cases of 'trenches' excavated in front of suction anchors for semi-taut deepwater systems
- Removal of soil in front of anchor reduces capacity and affects system performance, but how much?
- Systems require reassessment to determine extent of issues, and address potential mitigation
- Some systems now being designed with 'a priori' trenching





Ref: Bhattacharjee et al (2014)

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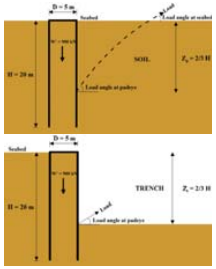
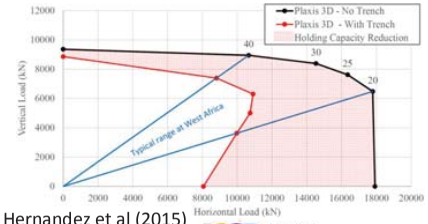
Suction anchors - trenching

- Refined 3D FEA helps understand failure mechanism and reduction in capacity
- Effects depend on load angle and geometry
- Example here for GoG soil
- Anchors may be 'saved' due to earlier conservatism in design (soil parameters, analysis method)
- JIP «TreMOOR» to investigate causes of trenching and investigate mechanisms

Diameter	5 m
Length	20 m
Padeye	13.3 m
Weight	900 kN
suC	2.4 kPa + 1.57 z
suD/suC	0.83
suE/suC	0.8





Ref: Hernandez et al (2015)

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Some conclusions

- Assessment of installed structures, foundations and anchors provides many opportunities as well as technical challenges
- Use of information from actual installation and improvements in practice will give a more reliable understanding of the performance
- Specific improvements to understanding of pile axial capacity and lateral load displacement response have given step-changes in industry practice
- Suction anchor assessments for life extension can take advantage of big developments in analysis methods and understanding cyclic soil behaviour
- Trenching in front of suction anchors needs better industry understanding and cooperation

Thank you

Any questions?

Chapter 10

Session 8: Inspection Planning with Respect to Crack Control

10.1 Presentation & article by Ole Tom Vårdal & Torgeir Moan

NTNU Norwegian University of Science and Technology

International Association of Oil & Gas Producers

Lessons Learned from Predicted Versus Observed Fatigue Cracks of Offshore Steel Structures in the North Sea

Ole Tom Vårdal, Torgeir Moan
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The 3rd Offshore Structural Reliability Conference
OSRC2016
14-16 September, Stavanger, Norway

Agenda

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- The challenge is to get a reliable estimate of the fatigue crack growth potential
- Validation study of Jacket structures – main conclusions
- Number of inspections and cracks detected in Jackets and Semi-Submersibles
- Scheduling the Improvement Maintenance and Inspections:
  - Method
  - Examples identifies the critical success factors
- High fatigue crack growth potential and the need for a larger reconstruction
- Efforts required to get estimate of the fatigue crack growth potential representing the frequency of occurrence

The key question!

AXESS

- How can we be sure that the facilities are sufficiently safe?
- The challenge of verification – assessment method and experience data

Can we trust;
a) the structural analyses ?
b) the fabrication quality ?
c) the technical documentation?
d) the inspection process ?

Detect -> Compensating measures

NORSOK Requirement and Jacket validation study

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NORSOK N-005 requirement:

Annex C.2 Jacket integrity

- To predict and to determine with a **reasonable level of confidence** the existence, extent and consequences of deterioration, damages and defects of the structural components are essential for decisions to maintain structural integrity of a jacket structure.

Study from 1991-99 – Validation of the tool for prediction

- 90% of the available inspection history for the Jacket structures in the Norwegian part of the North Sea

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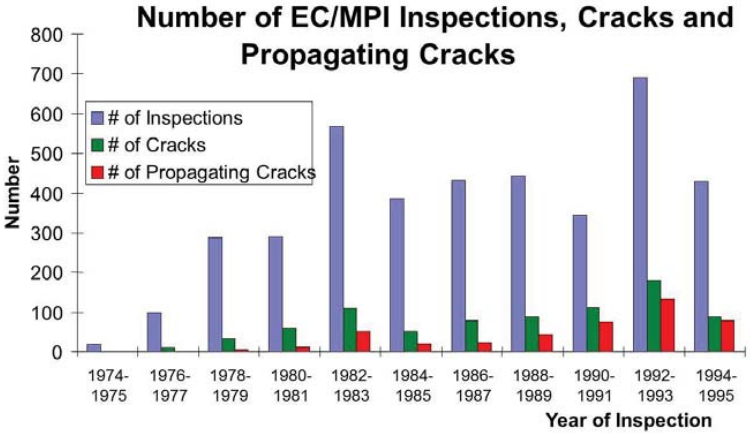
Jacket Validation Study



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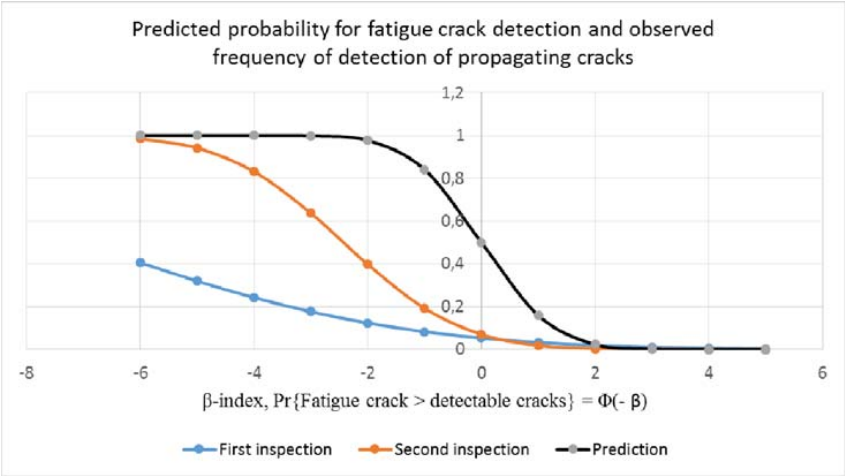
Jacket Validation Study



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Jacket Validation Main Conclusions

- The theory predicts 3 to 20 times more fatigue cracks than were observed.
- Between 10-40% of the fatigue cracks were detected at hot spots, which the validated probabilistic analyses approach would not have scheduled for inspection.
- There were observed correlation between fatigue life estimates and frequency of fatigue crack occurrence for the larger group of observation data. For larger sub-groups correlation were not observed.
- The estimated fatigue crack growth potential including the results from the previous inspection, is more closely related to the observed crack occurrence than for hot spot without previous inspection results.

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Collected experience data from 40 jacket structures in the North Sea. A total of 3366 NDE inspections.

	Non propagating cracks	Potentially propagating cracks	Most likely propagating cracks	Total number of cracks
Number of cracks	228	124	159	511
Percentage of inspections with cracks	6,77 %	3,68 %	4,72 %	15,2 %

Collected experience data from 12 semi-submersibles with more than 20 years of operation time in the North Sea. A total of 22282 NDE inspections.

	Non propagating cracks	Potentially propagating cracks	Most likely propagating cracks	Total number of cracks
Number of cracks	205	324	622	1164
Percentage of inspections with cracks	0,92 %	1,45 %	2,79 %	5,22 %

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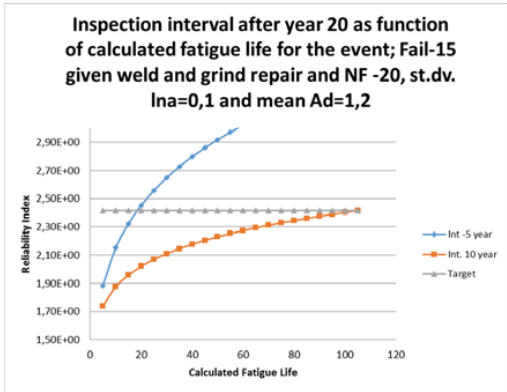
Scheduling the Improvement Maintenance and Inspections for Semi-Submersibles

Method

Target reliability, β =2.4
Pr{Failure} = Φ(- β) = 8E-3

Pr{Fatigue Crack > Detectable Crack} in the range <3%, 15%>



- Improvement required for Calculated fatigue life less than 19 years
- Inspection interval of 5 years for calculated fatigue life between 19 and 105 years

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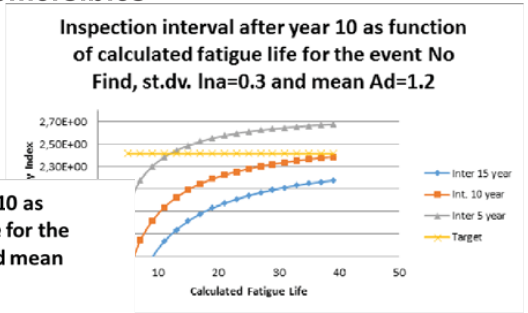
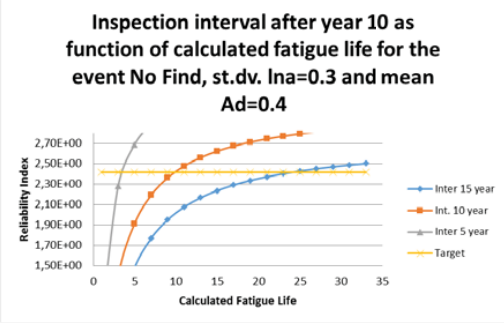
Scheduling the Improvement Maintenance and Inspections for Semi-Submersibles

Method

Mean POD = A_d= 1.2mm or 0.4mm crack depth, i.e. 90% confidence of crack length less than 37 and 12.5mm.





Modification/Improvement required for hot spot with calculated fatigue life less than 3.5 and 12 years. Limit for 5 year inspection intervals are 10 and 46 years respectively.

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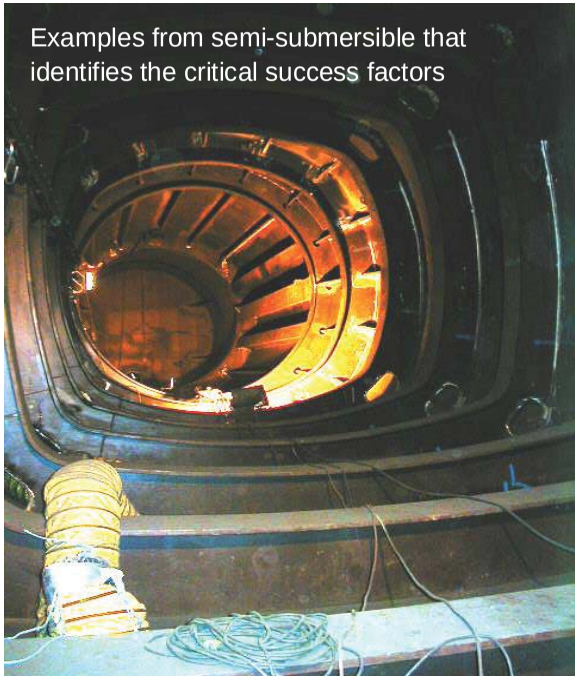
Scheduling the Improvement Maintenance and Inspections for Semi-Submersibles

Method

The important uncertainty elements:

- The quality in the inspection, repair and modification history
- Knowledge of the quality of the inspections done
- Knowledge of the quality of calculated characteristic fatigue load
  - Fatigue analyses methodology
  - Local design and fabrication quality – class of SN-curve
- Operation history and experienced fatigue loading

Examples from semi-submersible that identifies the critical success factors



A view inside the Horizontal bracing

- A total of 500meter had calculated fatigue life between 5 and 40 years
- 100% EC and/or MPI inspection after 10 years in-service
- 70% UT - inspection after 10 years in-service
- No fatigue cracks detected

Challenge:

Document remaining Service life of 20 years

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Crack occurrence in the VFB Semi-Submersibles in the period 2000 to 2011

Part of structure	*Type of crack*	*Number of cracks*
Columns	- Cut-outs in deck plates for pipes - Cut-outs in deck plates for vertical stiffeners - Various stiffener plate Intersections - Heavily corroded area in deck - Notch in vertical stiffeners	18 3 9 1 1
	Total cracks in columns	*32*
Main braces	- Stiffener ends - Stiffener to frame terminations	16 1
	Total cracks in braces	17
Brace to column intersection	-Vertical weld between brace and column - Horizontal weld between brace and column. - Gusset plates inside columns - Frame inside column	Total 10
Pontoon to sponson welds	Intersections of longitudinal butt welds with frames in pontoons	12
	Total number of cracks	73

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Case studies to demonstrate the effect of different follow up regime

Comparable stress level for unit 1 and unit 2:

The nominal stress level in the braces have an amplitude as high has 148 MPa. Larger area of nominal stress amplitude above 120MPa. For SN-curve D for air and stress concentration factor SCF= 1.0 we have characteristic calculated fatigue life of 20 and 47 years for these stress levels.

Unit 1 - The improvement scope had site follow-up by integrity engineers	Upon 10 years of operation after implementation of the improvement scope there were detected a total of 73 fatigue cracks of 1 at the weld improved area
Unit 2- Traditional follow-up team	Upon 5 years of operation after implementation of the improvement scope there were detected a total of 180 fatigue cracks at the weld improved area

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The Critical Success Factors

The different level of refinement for the follow-up regime.
Level 0 – basic follow-up regime defined in the design phase
Level 1 – extension of level 0 by use of updated fatigue analyses
Level 2 – use of a probabilistic approach in which the results from previous inspections are used to estimate the fatigue crack growth potential
Level 3 – an extension of level 2 that also includes the process of verification of the quality in: ○ hydrodynamic and structural analyses ○ construction work ○ in-service inspection ○ in-service integrity engineering ○ validation of the method for estimation of the fatigue crack growth potential

The need for a larger reconstruction

Target reliability, β =2.4, Pr{Failure} = $\Phi(-\beta)$ = 8E-3
Pr{Fatigue Crack > Detectable Crack} in the range <3%, 15%>

The number of MPI/EC inspections and number of cracks detected for different periods.

Year detected	Non propagating cracks	Potentially propagating cracks	Most likely propagating cracks	Num. of insp.
1998-2007	26	36	18	451
2008-2010	37	60	27	1014
2011-2014	4	25	35	937

Characteristics of detected cracks given in above presented as a percent of inspections done.

Year detected	Non propagating cracks	Potentially propagating cracks	Most likely propagating cracks
1998-2007	5.76%	7.54%	3.99%
2008-2010	3.65%	5.92%	2.66%
2011-2014	0.43%	2.67%	3.74%

The need for a larger reconstruction

Lessons learned, 1 of 3

- To maintain the technical integrity is not limited to maintain the given design.
  - In-service observations and root cause analyses will normally detect shortcomings in present design. Program for improvement to be defined.
  - Looking beyond present findings and update the follow-up scheme
  - Be aware of gross error in the input data for fatigue analyses and analyses for estimating the fatigue crack growth potential. E.g. mismatch between structural drawings and as-build structure

Lessons learned, 2 of 3

- The theoretical approach without awareness of the importance of As-Is mapping will most likely fail to give reliable estimate of the fatigue crack growth potential. The inspection, repair and modification history is the Key Book.
  - Through verification activities identify shortcomings in the theoretical analyses
  - Define inspection program for collection of experience data to be used for verification and validation

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Lessons learned, 3 of 3

- Secure correct competence for the different activities - including integrity engineering which is not always appreciated resource on site
  - The inspection process is the activity to collect and report observations. It is not to make conclusions related to technical acceptance.
  - The integrity engineering process is to define the need for information, improvement maintenance and report the safety level for structural integrity. To make conclusions for technical acceptance is a delivery of the integrity engineering process.
  - To define the deliveries and competence requirement for the inspection and integrity process, is essential.

AXESS

Scheduling the Improvement Maintenance and Inspections for Semi-Submersibles
Method

The important uncertainty elements:

- The quality in the inspection, repair and modification history
- Knowledge of the quality of the inspections done
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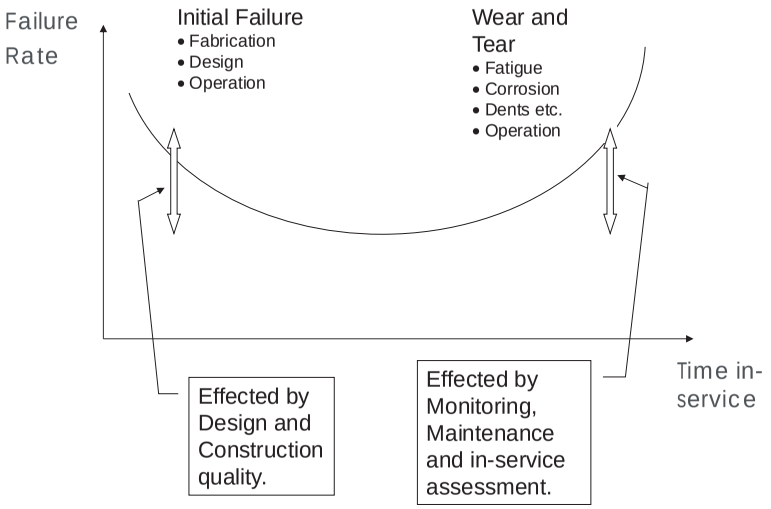

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Lessons Learned from Predicted Versus Observed Fatigue of Offshore Steel Structures in the North Sea

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ABSTRACT: In Structural Integrity Management (SIM), it is essential to know the fatigue crack growth potential. In the design phase generic information about the load effects and resistance and the inspection quality, possibly including measures about the inherent uncertainties, is used as a basis for design criteria and the inspection plan. In this paper it is emphasized that a rational SIM should also be based on a systematic use of information collected for the relevant structure during fabrication and operation. The need for and outline of a systematic lifecycle SIM is demonstrated in this paper based on lessons learned from refined fatigue analyses, fracture mechanics and probabilistic methods for offshore-structures in-service, and the importance of using information collected during fabrication and operation of the specific platform. The inspection history of more than 20 000 NDT inspections and detection of close to 1000 fatigue cracks serve as basis for the present study. These in-service data are used to assess how well we are able to quantify the fatigue crack growth potential, i.e. the capability to predict where and when fatigue cracks occur. The essential lessons learned are described and discussed. In particular, it is emphasized that a proper SIM involves assessment of possible structural modifications and scheduling of inspections with due account of the quality of the structure (i.e. As-is geometry - that can deviate from that of the design, observed cracks, possible corrosion conditions differing from the initial plan) and the inspection and repair process, and update of wave conditions.

KEY WORDS: Structural Integrity, Fatigue, Inspection, RBI, Probabilistic Analyses, SRA, Bayesian updating.

1 INTRODUCTION AND BACKGROUND

1.1 Introduction

This paper mainly deals with lessons learned related to the structural integrity management and the fatigue crack growth. Field experience in the period 1975 to 2016 have resulted in a change from a follow-up regime based on craftsman tradition and experienced engineers' judgement to some quantitative structural integrity management methodologies [20], [21]. Guidelines and regulations regarding the fatigue limit states were developed according to the improved knowledge from laboratory tests and more detailed methods to identify the fatigue loading [16].

The offshore industry early recognized that design took place under uncertainty and adopted risk and reliability methods to make more rational decisions. Structural reliability methods have been used to ensure that ULS requirements are consistent with the desired target safety level, to calibrate the safety in design codes to a certain reliability level [25], [26] and [27]. In addition to the uncertainties affecting the predicted behavior under extreme and fatigue conditions, inspection is subjected to uncertainty. Reliability methods are, hence, crucial to support decisions about safety and economy of degrading structures. Significant developments of structural reliability methodology, including Bayesian updating techniques, have taken place since the 1980's, [28], [29], [30], [31], [32], [33], [34], [35], [36] and [37].

While current design and inspection procedures have been established on a reliability basis of the generic information available at the design stage, it is important that information obtained during operation of individual structures, e.g. by inspections during operation, is used to update the inspection plan.

Probabilistic Inspection Analyses (PIA) used since the late 1980s, also include the information from in-service inspections and repair history for the calculation of the fatigue failure potential for a given hot spot e.g. [6], [12], [13], [14] and [15].

In welded structures, fatigue cracks almost always start at a weld defect. It is well known that sometimes the fabrication tolerance criteria are not fulfilled [18], [19]. The accuracy of the probabilistic forecast models depends on the quality of the information from the construction and inspection processes. Hence, the capability of a probabilistic forecast model for providing appropriate results is related to the whole structural integrity management process. This paper addresses guidelines for how to use a probabilistic forecast model in this perspective or more the need for guidelines for implementing, verification and validation of a probabilistic forecast model for fatigue. The summary of lessons learned relating to integrity management will be highlighted. Quantitative study results will be presented in separate papers.

A project to validate the PIA (probabilistic inspection analysis) approach for jacket structures, was conducted during the period 1994 to 1999 [1], [2], [3], [8], [39] and [40]. Section 1.2 and 1.3 summarizes the conclusions from this validation study and is used as reference in our considerations related to our capability to reliably forecast the fatigue crack growth potential for semi-submersibles.

The outline of this paper is as follows; Section 2 deals with the decision model for scheduling inspections, improvements and modifications. The sensitivity of the decisions to the

quality in the input data are discussed. Section 3 highlights important elements to be included in a structural integrity management process, and guidelines for how to ensure conservative forecast values of the fatigue crack growth potential are given. Section 4 gives the concluding remarks.

1.2 Experience with cracks in North Sea jackets

The inspection and crack detection histories of about 40 jacket structures were used to identify the degree of correlation between theoretical predictions and observed fatigue cracks by expressing predictions in probabilistic terms considering normal uncertainties, i.e. the probabilistic forecast of the fatigue crack growth potential by Akers tool for PIA analyses [12], [15], see Appendix A for model description. The conclusions from the validation study for jacket structures are as follows [1], [40]:

a) The theory predicts 3-20 times more fatigue cracks than were observed when information from previous inspections are not included.
b) Between 10-40% of the fatigue cracks were detected at hot spots, not scheduled for inspection according the probabilistic analyses approach
c) Correlation was observed between fatigue life estimates[1)] and frequency of fatigue crack occurrence for the majority of the observation data. For sub-groups, such a correlation was not observed.
d) The estimated fatigue crack growth potential[1)] including the results from the previous inspection is more closely related to the observed crack occurrence than for a hot spot without previous inspection results.

The predictions or forecasts obtained from the PIA analyses are presented in terms of the reliability index β_{dc}, which relate to the probability of fatigue crack detection by $P_{dc} = \Phi(-\beta_{dc})$, where $\Phi(.)$ is the standard normal distribution, see column 1 and 2 in Table 1. Table 1 shows the number of inspections and observation results grouped according to the forecasted index β_{dc} for the given inspection. Only the first-time inspection results are shown in Table 1, and the value of index β_{dc} will only vary with calculated fatigue-life estimate and number of years in service before the inspection is done. Previous inspection or repair at a considered hot spot will also affect the forecasted index β_{dc}. The number of fatigue cracks detected shown in column 5 in Table 1 will vary with how we classify the observed cracks in the group of fabrication defect or fatigue crack. Column 4 expresses the forecasted or predicted number of fatigue cracks to detect and is the product of the numbers in columns 2 and 3. The ratio between the numbers in columns in 4 and 5 is the basis for conclusion a) above. The ratio between the number of fatigue cracks observed before the β_{dc}-value reach the limit for inspection scheduling and total number of detected fatigue cracks in column 5, is the basis for conclusion b) above. As discussed in Section 2.2 and 4.3, the limit β_{dc}-value for inspection scheduling is between 1.0 and 1.9 for fatigue failure without substantial consequences, i.e. the corresponding probability for fatigue crack detection is between 3 and 15 per cent ($\Phi(-1.9) = 0.03$ and $\Phi(-1.0) = 0.15$).

Conclusions c) and d) are based on the results shown in Figures 1a and 1b. The intention with Figures 1a and 1b is to visualize the deviation between observations and theoretical forecast. $\Phi(-\beta_{dc})$ indicates the probability of detecting a fatigue crack and is presented in Figure 1a as function of the forecasted increment of β_{dc} together with the regression line of observed frequency of detection. Figure 1b shows the same curve as in Figure 1a but by using betta value also for the vertical axis i.e. $\beta^0_{dc} = \Phi^{-1}$(Observed frequency of fatigue crack detection). The curves presented in Figure 1a and b are:

a) "Predicted" – (Column 2 in Table 1) This curve represents the theoretical prediction, i.e. expected frequency of fatigue crack detection when inspections are done at given β_{dc} - value (β_{dc} – value is given in column 1 in Table 1). For accumulated fatigue damage of 0.1, 1.0 and 3.0 at time of inspection the β_{dc} - value and probability for detection of fatigue crack is 1.8, 0.4, -0.3 and 4%, 35%, 64% respectively.
b) "First Inspection" – (Regression line for the numbers in column 6 in Table 1) This curve represents inspection events where no previous inspections have been performed
c) "Second Inspection" – This curve represents inspection events when previous inspection has been performed

Table 1 The first column contains the forecasted reliability index $\boldsymbol{\beta_{dc}}$-value for the event of fatigue crack detection. Column 2 is the forecasted probability for fatigue crack detection of the inspections in the class of forecasted β_{dc}-value i.e. $= \Phi(-\beta_{dc})$. Column 3 gives the number of inspections performed within the class of forecasted β_{dc}-value. Column 4 contains the product of column 2 and column 3. Column 5 is the number of the inspections in column 3 with fatigue crack detection and is the sum of the given percentage belief that the detected cracks are due to fatigue. Column 6 expresses the ratio between column 5 and 3.

β_{dc} - value (PIA)	Estimated Probability for Fatigue Crack Detection $P_{dc} = \Phi(-\beta_{dc})$	Number of Inspect.	Number of Fatigue Cracks Predicted	Number of Fatigue Cracks Detected	Ratio Fatigue Cracks vs. Inspections
-4.5	0.999997	53	53	19	0.36
-3.5	0.9998	43	43	7	0.16
-2.5	0.994	101	100	19	0.19
-1.5	0.933	128	119	15	0.12
-0.5	0.691	242	167	28	0.12
0.5	0.309	416	129	49	0.12
1.5	0.067	858	57	16	0.02
2.5	0.006	707	4	15	0.02
3.5	2,33E-04	136	0	10	0.07
4.5	3,40E-06	39	0	2	0.05
5.5	1,90E-08	16	0	0	0
		2739	673	180	

Except for the curve "Predicted", this is the regression lines made based on the observations. The deviation between the curve "Predicted" and the curves denoted "First Inspection"

[1] The term fatigue life estimate and fatigue crack growth in the conclusion from [1] and [2] are respectively the estimated probability for detectable crack to be present at time of inspection for hot spot without and with previous in-service inspection included in the PIA analyses.

and "Second Inspection" shows the degree of correlation between observed frequency and theoretical prediction, [1]. The curve "First inspection" shows a larger deviation towards the "Prediction" or forecasted potential for fatigue crack growth. In Figure 1b we also see that curve "First inspection" cross the "Prediction" curve at β_{dc} = 2.1 that represent probability for fatigue crack detection of 1.8%. Hence, the general high degree of over prediction of fatigue crack detection by the curve "Prediction" change to be an under prediction for probabilities less than 1.8% of the first inspections to detect fatigue cracks. The model used to predict the fatigue crack growth is subjected to uncertainty due to:

- Fabrication defects in weld material
- Misalignment
- Poor local design
- The structural analyses have not assessed all hot spots i.e. omit hot spots that shows to have high fatigue crack growth resistance

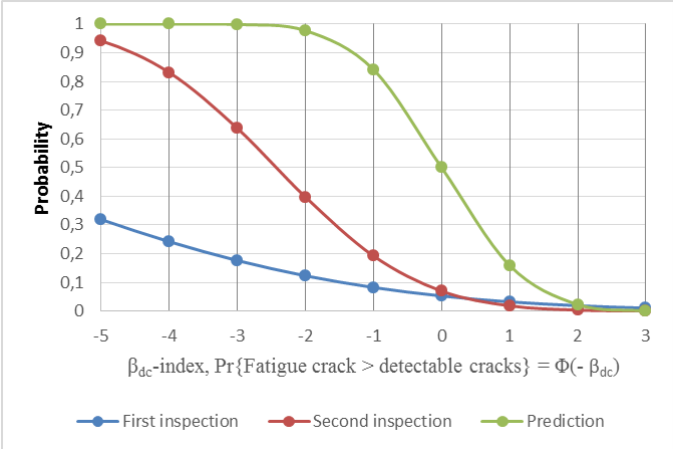

Figure 1a The Regression/Trend line of observed fatigue cracks as a function of the estimated reliability for fatigue crack detection, $P_{dc} = \Phi(-\beta_{dc})$ i.e. decreases with incrementing β_{dc}.

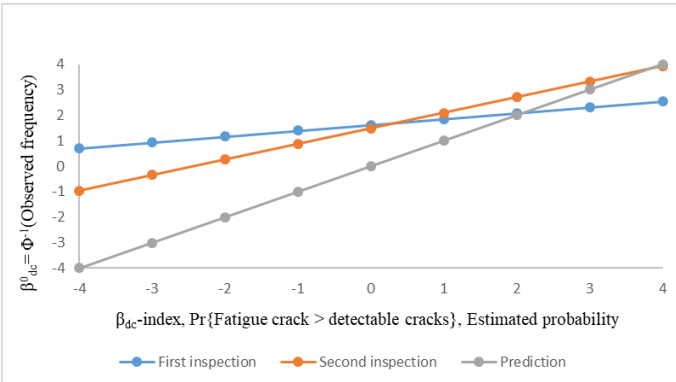

Figure 1b Same as Figure 1a and use of $\beta^0_{dc} = -\Phi^{-1}$(Observed frequency of fatigue crack detection) for the vertical axe. In a β-β diagram the prediction curve form the identity line. The curves form straight lines as the curve of first inspection and second inspection are made by linear curve fit in a β-β diagram for the observed frequency.

The experience with tubular joints in jacket suggests that there is a few per cent probability that fatigue cracks develop more or less independent of the reported characteristic fatigue life estimate. To reduce the inherent uncertainty the quality assurance in the design and construction phases needs to be improved, including the inspection process and re-analyses in the operational phase. Extended As-Is mapping not limited to hot spot with reported significant fatigue loading, is also a cost-effective measure to reduce the uncertainties in fatigue crack growth predictions for fixed and mobile offshore structures, and hence improve predictions by PIA analyses.

The curve "Second inspection" is closer to the curve "Predicted" and is generally on the safe side. The curve "First inspection" gives identical inspection scheduling with a deterministic model. The deviation between the curve "First inspection" and "Predicted" is therefore a measure of improvement by use of the Probabilistic Inspection Analyses tool PIA by including information from previous inspections. It is also noted that in the first inspection fabrication defects and improper local design can be detected and relevant measure taken, implying an improved quality of predictions. In this study the PIA tool of Aker, [12] and [15] was applied. However, it is believed that the conclusions made in the jacket validation study are general for use of the PIA methodology, [6] and [14].

Table 2 shows the number of cracks detected and their classification according to the character of the cracks, i.e. whether they are propagating or not. The term "propagating cracks" is used since fatigue crack growth in welded structures starts to grow from initial defects. Frequently, the cracks start to grow from initial defects that may be classified as gross errors, i.e. an initial crack depth larger than normal acceptable fabrication quality. The observed cracks in the validation study for jackets were used to evaluate initial crack depth and Probability of Detection (PoD) value for the inspection methods [8]. The study of 360 propagating cracks and 330 non-propagating cracks in tubular joints reported that the most likely initial crack depth was close to 1mm for both groups of cracks. The mean PoD value based on the in-service observations for inspection below water on jacket structures was found to be 1.95mm.

The probabilistic inspection analyses were based on fatigue analyses. The fatigue analyses methods used in 1990 to 1994 differ from current methods. The conclusion of the validation of 1999 will therefore need to be adjusted for changes in the fatigue analyses methodology.

Table 2 Summary of the collected experience data used for the jacket validation study in [1], from 40 jacket structures in the North Sea. A total of 3366 NDE inspections have been reported.

	Non-propagating cracks	Potentially propagating cracks	Most likely propagating cracks	Total number of cracks
Number of cracks	228	124	159	511
Percentage of inspections with cracks	6.77 %	3.68 %	4.72 %	15.2 %

1.3 Adjustment of PIA model to obtain improved correlation between predictions and observations in North Sea jackets

A mathematical optimization of the basic variable for the PIA model to secure best possible fit between predictions and observations was conducted [1] and [39]. Appendix A describe the PIA model used for the Jacket validation study [15] and Table 3 shows the adjustments done [39]. Figure 2 shows the curve fitted to the observations of first inspection, the predictions by PIA by use of the input data presented in Appendix A and the predictions by PIA using the adjustment presented in Table 3 i.e. the curve denoted "Model 0" representing the PIA model presented in Appendix A. The adjustment values of POD= λ and A0=λ_0 are from [8].

Table 3 Adjustment in basic variable securing best possible fit between prediction and observation. Where Y is correction factor on the geometry function, M is the exponent in the stress range, A is the Weibull scale factor, λ_0 is the mean value of initial crack size and λ=POD is the mean value of crack depth for detection by MPI/EC. X is the value of Ln(A) corresponding to characteristic calculated fatigue life and will typically be in the range 1.9 to 2.5 i.e. characteristic fatigue life of 300 and 5 years respectively for the Weibull shape parameter of 1.0.

Model	Mean value of Y	COV[Y]	M	Mean value of Ln(A)	St.dev. of Ln(A)	λ_0 [mm]	λ [mm]
0(Base case)	1.0	0.1	3.1	x	0.246	0.11	1.2
1	1.0	0.1	3.1	x	0.246	0.36	1.95
2	0.78	0.55	2.9	0.669x	0.0803+0.0491x	0.36	1.95
3	0.90	0.63	3.34	0.769x	1.15 ·(0.0803+0.0491x)	0.36	1.95
4	0.813	0.651	3.06	0.769x	1.15 ·(0.0803+0.0491x)	0.326	1.85

The deviations between the Model 4 and PIA model i.e. Model 0, are to be given a physical interpretation and further studies required. The applied PIA model used in the jacket validation study give results in accordance with the recommendations in /6/ and fatigue analyses methodology classified with st.dev. Ln(A) of 0.2. (Note, that the PIA model used for the Jacket study and recommendations in /6/ include a correlation between the Weibull shape and scale parameter that not are include in the PIA model used for Semi-submersible presented in the following sections. See Figure 5.)

To obtain best possible fit between the results from more than 3000 MPI/EC inspection and forecast by the PIA model larger modifications in the model are required. Hence, the model recommenced by /6/ will also require larger modification before it provide forecast of fatigue crack growth in line with reality.

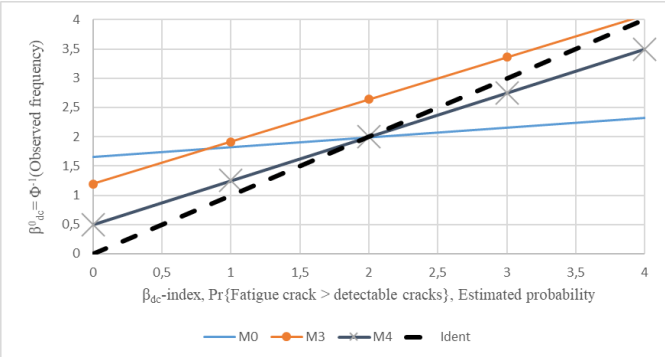

Figure 2 Comparison of observed and predicted reliability index for fatigue crack detection at first time of inspection for original M0 and adjusted models M3 and M4 as given in Table 3. Use a β-β diagram as in Figure 1b and limited to β > 0.

1.4 Field data from semi-submersibles

The data used in this paper were obtained from 12 floating offshore units. They are of semi-submersible design and each has more than 20 years of operation in the North Sea. For these 12 units we have collected the results of more than 20000 MPI/EC inspections and a detailed description of close to 1000 fatigue cracks, see Table 4. Moreover, the experience data also includes the assessment and analyses of more than 25 onshore yard stays for modifications and/or renewal survey. Normally re-analyses of the global and local FLS (Fatigue Limit State) analyses are done for each yard stay. These data are available to the authors, but have not yet been made publicly available. We have not done systematic quantitative comparisons between observations and predictions in the same manner as made in the validation study for jackets.

Table 4 Summary of the collected experience data from 12 semi-submersibles with more than 20 years of operation time in the North Sea. A total of 22282 NDE inspections have been reported.

	Non-propagating cracks	Potentially propagating cracks	Most likely propagating cracks	Total number of cracks
Number of cracks	205	324	622	1164
Percentage of inspections with cracks	0.92 %	1.45 %	2.79 %	5.22 %

2 DECISION MODEL FOR SCHEDULING INSPECTION, IMPROVEMENTS AND MODIFICATIONS

2.1 Introduction

It is well known that design fatigue analyses are subjected to significant uncertainties. This uncertainty might be reduced based on observations through inspections. A probabilistic fracture mechanics model makes it possible to obtain an updated estimate of the fatigue crack growth potential, i.e.

updated by use of information from the in-service inspection, [6], [12], [13], [14] and [15].

Figure 3 schematically illustrates how information from inspection and fatigue analyses are combined for a specific hot spot location. The upper part of the figure illustrates the simulated crack size distribution and the lower part illustrates how the estimated probability of a fatigue failure develops. This illustration is based on an inspection with the result of "No Find", i.e. the event of a crack smaller than the detectable crack size, is used for the probability update.

After updating, the estimate of the fatigue crack growth potential is improved. Updating based on an inspection result of detecting a fatigue crack, will increase the probability of fatigue failure and not reduce the fatigue failure probability as shown in Figure 3.

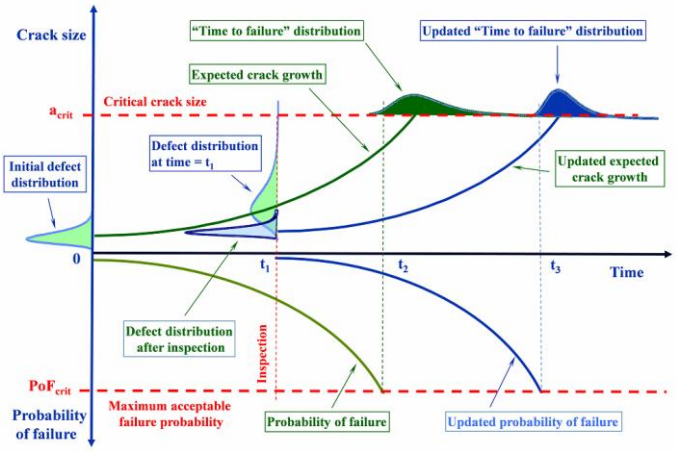

Figure 3. Schematic illustration of crack growth and probability of fatigue failure before and after an inspection. [6]

The philosophy and methodology described in [4] and [5] have since 1999 been applied for scheduling inspections, improvements and modifications for offshore floating structure. (See [4] and [5] for the minor deviations between model used for Jacket structures presented in Appendix A and PIA model used for floaters since 1999.) The probabilistic fracture mechanics model used, gives results that are comparable with the probabilistic fracture mechanics model recommended in [6], see Figure 4. The curve "DNV Curve" is from [7] but is very close to the curve in Figure 8-1 in [6] for CoV=0.2 related to load effect. Figure 5 contains a comparison with curves in Figure 8-1 in [6].

The PIA-analyses carried out are based on the long-term distribution of the fatigue load effect i.e. the Weibull scale and shape parameters Ln(A) and B, respectively. The fracture mechanics crack growth model used in the PIA analysis is calibrated against a selected SN-curve e.g. SN-curve F for cathodic protected steel [7]. Based on the reported characteristic fatigue life for each hot spot and the SN-curved used for the calibration of the PIA model, the mean value of Ln(A) is calculated. Without specific knowledge of the value of the Weibull shape parameter B, the default value of 1.11, is used [4], [12] and [15]. Hence, all uncertainties related to the SCF (stress concentration factor), notch stress effects and selection of SN-curve are related to the Weibull shape parameter Ln(A). The uncertainties included in the SN-curve are by calibration included in the other variables in the fracture mechanics crack growth model. The adjustments of the probabilistic forecast model presented in section 1.3 included large changes for the fracture mechanics crack growth variable calibrated with a SN-curve, [1], [39] and [40].

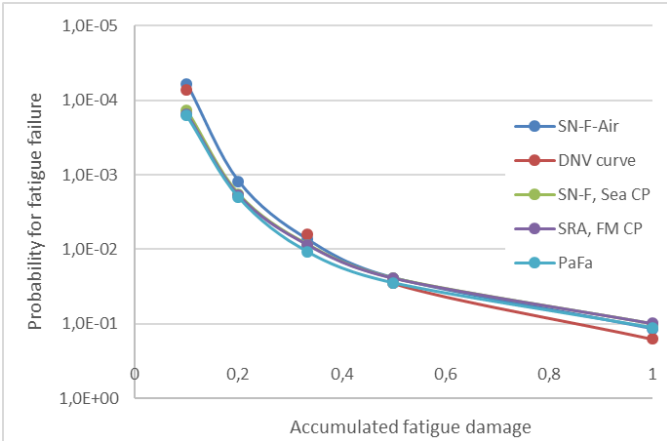

Figure 4. Different curves for estimating the probability of a through thickness fatigue crack (TTC) as function of accumulated fatigue damage = years in service/calculated characteristic fatigue life. The curve "SN-F-Air" and "SN-F, Sea CP" are probabilistic models based on SN curve F for air and CP protected, respectively. The "DNV curve" is from [7] and "SRA, FM CP" is from the model used in [4] and [5]. The curve "PaFa" is the latest model used for results presented in this paper.

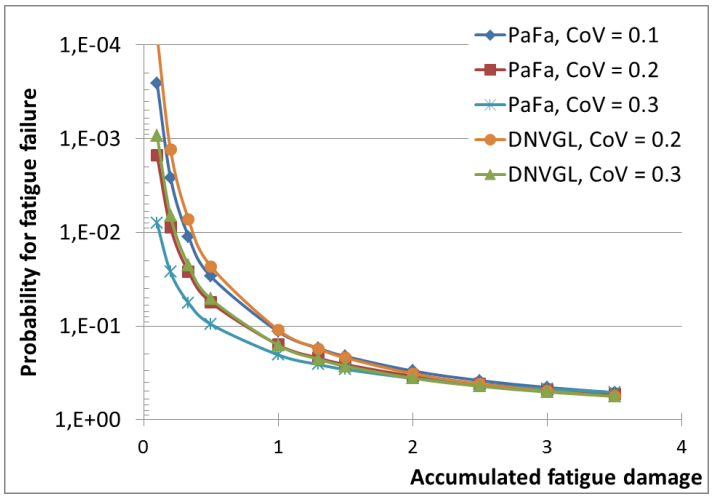

Figure 5. Comparison of probability of fatigue failure (TTC) between the PaFa model used since 2004 and the new model presented in [6] from 2015. The "PaFa" model does not include correlation between Weibull scale and shape parameter for the distribution of load effect. This effect is equivalent to a shift in the uncorrelated model of around 0.1 for the CoV-value of the load.

The curves presented in Figures 4 and 5 are initially considered as notional probability levels. To make these

probabilistic fracture mechanics models to represent a more true frequency of occurrence, they need to be validated by field data, that is by comparing observed and predicted frequency of fatigue crack occurrence. Ref. Section 1.2 and 1.3 and reported deviations between predictions and in-service observations.

2.2 The challenges of establishing and use of a forecast model

The PIA models described above are probabilistic forecast of fatigue crack growth. They present probability estimates of where and when fatigue cracks will occur. Figure 6 shows the building blocks and activities for establishing of a PIA model. To be used as a building block in the SIM process.

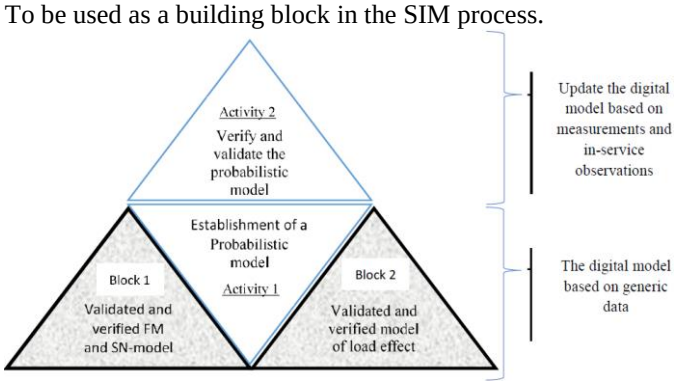

Figure 6 The probabilistic forecast model for the event fatigue crack occurrence is an important building block in the Structural Integrity Management and is established by building block 1 and 2 as well as activity 1 and 2.

Building block 1 represents the physical nature of resistance to fatigue crack growth. Building block 1 deliveries are fracture mechanic models or SN-models developed and validated based on laboratory tests. Building block 2 provides models for fatigue load effects i.e. structural and hydrodynamic analyses validated by model tests and measures on real structures. The studies of uncertainties and mathematical modeling of the forecast model is an activity for establishing a probabilistic forecast based on the input from building block 1 and 2. Before the probabilistic forecast model of fatigue crack growth can be applied as a building block in the SIM process a verification and validation activity is proposed i.e. activity 2 in Figure 6. Ref. [6] is a detailed description of activity 1 in Figure 6. Activity 2 in Figure 6, are not properly described in regulations, guidelines or recommended practice known to the authors.

In the field of e.g. geophysics however, validation of the forecast model through comparison with observations is essential in order to trust the outcomes of the models. The results from the jacket validation study, especially for the event first inspection presented in section 1.2 and 1.3, identify the need for activity 2 in Figure 6.

2.3 Comments related to target level

It is challenging to establish a target level for acceptable fatigue crack growth potential when notional probability estimates are applied. This challenge is discussed in [2] and [17], and for the decision of reconstruction described in [4] and [5] we applied a target case and not a universal target value. The design fatigue factor (DFF) of 3 and 10 (or fatigue damage of 0.33 and 0.1) given for different failure consequence class and no account of in-service inspections as discussed in [2] and [17], may be used for calculation of a target value. The term target case is used when the same probabilistic model is used for establishing the target level as for calculating the actual failure probability at the hot spot considered.

It is highly recommended to use target cases and not a predefined generic target level. Moreover, special considerations should be made to model the basic variables that are not used in the determination of the target value, e.g Probability of Detection (PoD) value for the inspection methods as well as the variables related to changes after weld improvement by grinding.

Since 2004 we have applied a target case also including the basic variable of crack detection i.e. the PoD value. The traditional follow-up approach by the classification societies is applied. The target case is a hot spot given an estimated fatigue life of 20 years from traditional fatigue analyses and inspected by MPI every fifth year with the result “No Find”. Figure 7 shows the analyses results of a model denoted “DNVGL, CoV=0.2” in Figure 5 and the inspection history of “No Find” every fifth year. The target level in Figure 7 is 5.0E-3 and by use of the model denoted “PaFa, CoV=0.1” in Figure 5 the target level will be 7.8E-3 for this target case. As discussed in [42] the calculation of target levels is highly effected by the selected uncertainty level in the fatigue load effect.

The semi-submersibles operating in the North Sea have leak detection system. Even for the most critical hot spot in the hull the leak detection will be triggered weeks before the crack reaches a critical size for rupture and threatening the structural integrity. Moreover, the amount of leakage from a crack can easily be handled by the ballast system. All hot spots are classified with consequence class DFF=3 and a have recommended target level of 7.1E-3 in [6].

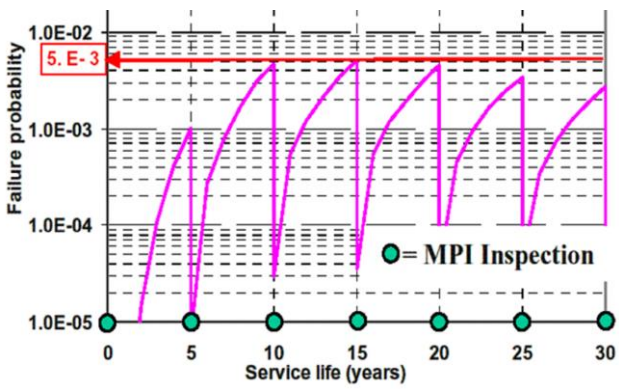

Figure 7 Illustration of the estimated probability of fatigue failure as a function of time in service for a hot spot with calculated characteristic fatigue life of 20 years and NDE every fifth year with result “No Find”.

2.4 A Case demonstrating the scheduling process for improvement maintenance and inspections

This case study refers to a semi-submersible with 1055 hot spots to be scheduled for the in-service follow-up program by use of the PIA approach. Table 5 summarizes the calculated characteristic fatigue life for these 1055 hot spots. If not specifically stated the presented calculated fatigue life are characteristic fatigue life and not mean fatigue life or any adjustment based on service time or design factors.

The characteristic fatigue life estimates presented in Table 5 are based on traditional design fatigue analyses and As-Is mapping [6] and [7]. The As-Is mapping describe the local design, quality of weld as well as surface condition as coated or corroded i.e. all information required to define notch stress effects, SCF and SN-curve selection. For each hot spot the history of repairs, modification, coating condition, corrosion, etc. are used to evaluate changes in the value of Ln(A) for different time periods of operation. As mentioned above, the Ln(A) value used as input to the PIA analyses includes the effect of variation in notch stress, SCF and SN-curve selection. The selected semi-submersible has been in operation for more than 20 years and has close to 2000 records of reported observations used for considering the hot spots fatigue crack growth resistance and quality class of the characteristic structural fatigue analyses according the grouping used in [6]. These are in addition to the records from the NDE(Non Destructive Examination). For the given case, the design fatigue analyses are refined re-analyses representing the state of art made to ensure the best possible input to the PIA analyses. The preferred approach is to directly use the Weibull scale and shape parameter from the fatigue analyses. In principle, the estimation of fatigue crack growth resistance should be made after obtaining information after the as-is mapping. However, it is more convenient to report the fatigue load effect on the format of characteristic fatigue life estimate even if these generic estimates of the fatigue crack growth potential used in the design phase are historical values as soon as knowledge from construction and operation is available.

Table 5 The 1055 hot spot to be assessed by PIA analyses grouped by range of characteristic calculated fatigue life. The fatigue life estimates used for calculation of the Weibull scale parameter Ln(A). The relation between fatigue life and Ln(A) is here given by SN-curve D for air and Weibull shape parameter B=1.0.

Range of Fatigue life in years	Weibull scale parameter i.e. the Ln(A) value	# of Hot Spots within group	Cumulative list # of Hot Spots
1-5	2.59 – 3.07	0	0
5-10	3.07 – 2.85	5	5
10-15	2.85 – 2.73	5	10
15-20	2.73 – 2.64	4	14
20-30	2.64 – 2.53	19	33
30-40	2.53 – 2.45	50	83
40-60	2.45 – 2.34	128	211
60-100	2.34 – 2.20	132	343
100-300	2.20 – 1.93	205	548
300-∞	1.93 -	507	1055

Classification societies specify follow-up activities during operation based on 5 year intervals between the renewals of certificate, e.g. [22]. The scope for inspection and improvement maintenance during the renewal surveys are prepared within 1 year after the last performed renewal survey and schedule the first activity for a hot spot. Then hot spots are scheduled for inspection in the next renewal survey while other hot spots have a sufficient fatigue crack growth resistance to be in operation for at least 10 and 15 years before inspection is required. For hot spots that reach the target safety level before 5 years of additional service, improved fatigue capacity is required, normally by modifying the structural detail. Hence, the scheduling process group the hot spot considered into the following:

- To be modified for improved fatigue crack growth resistance
- To be inspected within 5 years additional service
- To be inspected within 10 years additional service
- To be inspected within additional service of 15 years or longer

It is impractical to perform separate analyses for each hot spot as shown in Figure 7. For the given case, it is of interest to consider the analysis of hot spots (with different fatigue load effect) that have been in operation for 20 years and the fatigue crack growth potential after 5, 10 and 15 years additional period of service. As mentioned above the fatigue life estimates from the design fatigue analyses are used to report the fatigue load effect and not the Weibull distribution and related shape and scale parameter. Hence, the forecasted fatigue crack growth potential shown in the following figures use the calculated characteristic fatigue life estimate on the horizontal axis and not the Weibull scale parameter Ln(A).

Figure 8 provides the required analysis results for the hot spots inspected after 20 years' service with the result "No Find". The figure shows the reliability index β and not probability for fatigue failure along the vertical axis. The relation between probability and reliability index is given by $P_{TTC} = \Phi(-\beta)$ where $\Phi(.)$ is the standard normal distribution and P_{TTC} is the probability of a through thickness crack that is defined as fatigue failure. The curves "5 years additional service", "10 years additional service" and "15 years additional service" in Figure 8 intersection with the target value for characteristic calculated fatigue life of 8, 23 and 40 years respectively. (If the Weibull scale parameter were used along the horizontal axis and not the characteristic calculated fatigue life, the intersection with the target value had been for Ln(A) value of 2.42, 2.19 and 2.07 using SN-curve D for air and Weibull shape parameter of 1 as reference for expressing the functional relationship between the two alternatives for reporting the long term distribution of the fatigue load effect. i.e. by characteristic fatigue life estimate and the value of Ln(A).) All hot spots with calculated fatigue life less than 8 years (i.e. Ln(A)>2.42) are to be modified to get improved fatigue crack growth capacity. All hots spot with calculated fatigue life between 23 and 8 years (i.e. 2.42 > Ln(A) > 2.19) are scheduled for inspection during the next renewal survey after 5 years' service. The hot spots with calculated fatigue life between 40 and 23 years (i.e. 2.19 > Ln(A) > 2.07) are

not to be inspected during the next renewal survey as their resistances against fatigue crack growth are sufficient for at least 10 years' service time without inspection. Hence, the first inspection is post bound to the renewal survey after 10 years in operation.

The method statement for scheduling the in-service program contains a database of analyses results on the format of Figure 8 for all combinations of inspection, repair, modification and service history as well as different combinations of uncertainty level in the fatigue load effect given as st.dev. of Ln(A) and quality of the NDE inspection as mean value of detectable crack depth $A_d = PoD_{Mean}$.

Figures 9, 10 and 11 show the reliability curves for hot spots with the following inspection and repair histories:

- Inspected after 15 years in operation with a through thickness fatigue crack (TTC) or close to a TTC. The crack was repaired to ensure a fatigue capacity as in the original design, including a weld improvement by burr grinding.
- After 20 years in operation the hot spot was inspected by MPI or EC with the result of "No Find"

Only the curves for the reliability against fatigue failure after 5 and 10 years additional service are shown, i.e. the curves denoted "5 years additional service" and "10 years additional service". The reliability curve in Figure 9 intersects with the target level for calculated fatigue life of 14 and 73 years. Hot spots with characteristic calculated fatigue life of less than 14 years are scheduled for improvement of fatigue capacity. Hot spots with characteristic calculated fatigue life of less than 73 years are scheduled for inspection during the next inspection campaign to start within 5 years additional time of operation.

Figure 10 is the same as Figure 9 but the inspection quality in terms of mean detectable crack depth has been improved from 1.2mm to 0.7mm. The intersection with the target reliability level has changed to 11 and 59 years calculated fatigue life, respectively. Figure 11 is the same as Figure 10 but the uncertainty level of the characteristic calculated fatigue life is increased, i.e. st.dev. of 0.3 for Ln(A) and not 0.1 as in Figures 8, 9 and 10. The intersection with the target reliability level has changed to 177 year of calculated fatigue life for reliability curve of 5 years additional service. The curve for 10 years additional service does not reach a reliability level as high as the target value. Before discussing the analyses results in Figures 9, 10 and 11 we need to consider the effect of large numerical uncertainties present in complicated inspection and repair histories.

Figure 12 is the same as Figure 9 but without the event fatigue failure detected after 15 years in operation, and in addition the uncertainty level of the characteristic calculated fatigue is increased, i.e. st.dev. of 0.3 for Ln(A), and not 0.1 as in Figure 9. The intersections with the target reliability level has changed to 7 and 15 from 14 and 73 years calculated fatigue life, respectively.

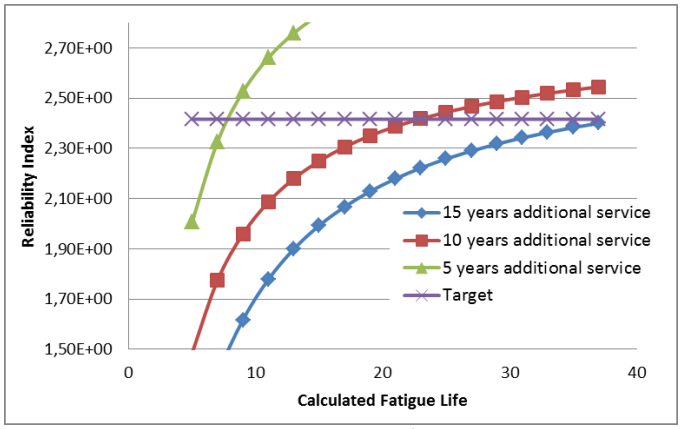

Figure 8 Reliability index β (= - Φ^{-1}(Probability for fatigue failure) where Φ(-) is the operator for the standard normal distribution) as a function of calculated fatigue life. The curves represent the reliability index at 5, 10 and 15 year additional service life after 20 years in operation and inspected with "No Find". The curves "5 years additional service", "10 years additional service" and "15 years additional service" intersection with the target value for calculated fatigue life of 8, 23 and 40 years respectively. The uncertainty level of fatigue load is given by a st.dev. of 0.1 for Ln(A) where A is the Weibull shape parameter for the long term distribution of the load effect. The quality of inspection is assumed to give mean detectable crack depth to be 1.2mm i.e. $A_d = PoD_{Mean} = 1.2$

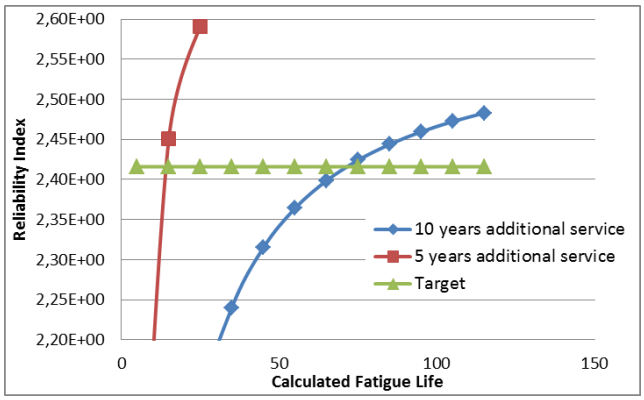

Figure 9. The reliability index as a function of initial estimated characteristic fatigue life for a given inspection, repair and modification history i.e. detection of fatigue failure after 15 years in operation, weld repair to original design and burr grinding for weld improvement and inspection event "No Find" after 20 years' service (5 year service after the repair). The two curves "5 years additional service" and "10 years additional service" give the reliability against fatigue failure after 5 and 10 years additional time of operation. The uncertainty level of fatigue load effect is given by st.dev. of 0.1 for Ln(A), where A is the Weibull scale parameter of the long-term distribution of the fatigue load effect. The inspection is done according to procedure and skills securing mean detectable crack depth to be 1.2mm, i.e. 90% confidence of detection of cracks of length 37mm.

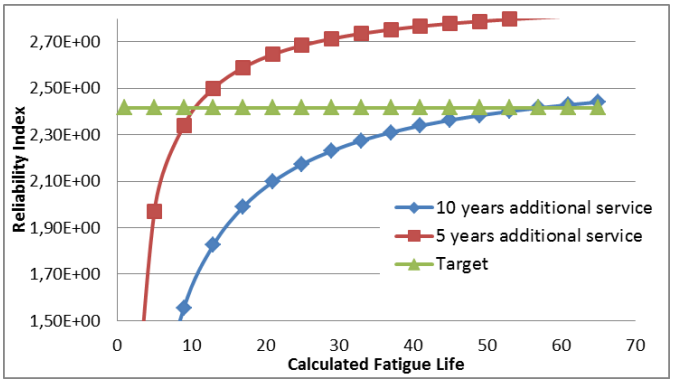

Figure 10. Same as Figure 9 but the quality of the inspection has been improved with a mean PoD value of 0.7mm of crack depth that represents a 90% confidence of detection of cracks of length 22mm.

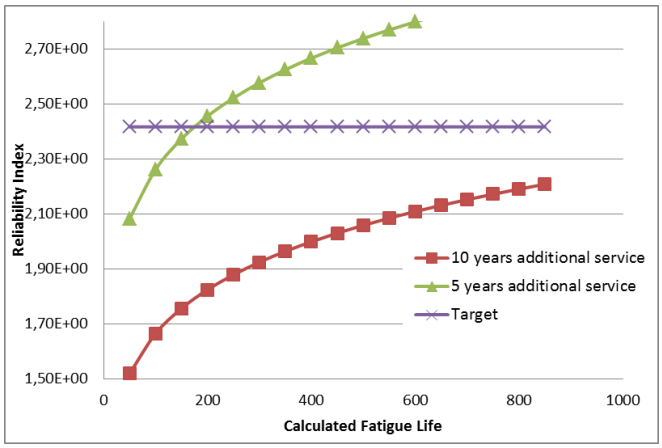

Figure 11. Same as Figure 10 but the uncertainty level of the estimated long-term distribution of the load effect has increased from std. dev. 0.1 to 0.3 for the Ln(A).

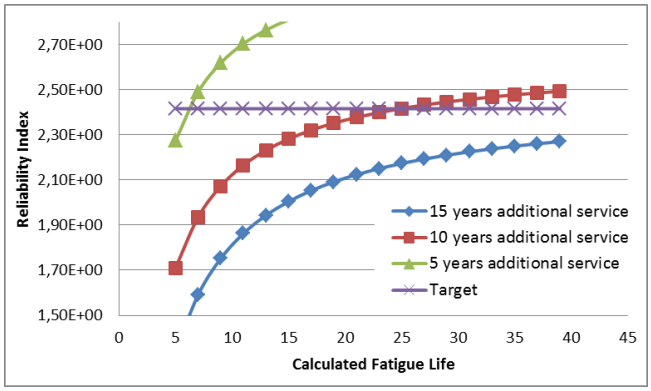

Figure 12. Same as Figure 9 but without the event of fatigue failure after 15 years' service as well as the uncertainty level of the characteristic calculated fatigue life is increased i.e. st.dev. of 0.3 for Ln(A) and not 0.1 as in Figure 9.

3 CONSIDERATIONS RELATING TO INPUT TO THE PROBABILISTIC FRACTURE MECHANICS MODEL, EXPERIENCE DATA AND EXPERIENCE DATA FROM SEMI SUBMERSIBLES

3.1 The selection of uncertainty level in the fatigue load effect and inspection quality, i.e. st.dev. of Ln(A) and mean PoD

The database of PIA analyses used during the scheduling of modifications and inspections as described above provide analyses results for combination of uncertainty level in the fatigue load effect and inspection quality i.e. st.dev. of Ln(A) and mean PoD. The database is established by experts and subjected to detailed quality assurance. However, the use of such a database of analyses requires proper training and guideline of use. The following topics are focused during the training:

1) Selection of PoD value and the sensitivity of the estimated time for next inspection due to change in the PoD value.
2) Selection of st.dev. of Ln(A) value and the sensitivity of the estimated time for next inspection for changes in st.dev. Ln(A).
3) The need for adjustment in the estimate of service year to next inspection or scope for improvement maintenance as a consequence of variation in fatigue load effect or fatigue crack growth resistance during time in service.

Sensitivity studies for hot spot with the inspection history of "No Find" shows a high negative sensitivity of the service time to the next inspection with respect to changes in the PoD, [42]. The sensitivity related to changes in the st.dev. of Ln(A) is relatively low for this kind of hot spot and inspection history of "No Find".

For a hot spot with inspection history of "fatigue crack detection and repair" shows a high negative sensitivity of the service time to next inspection with respect to a change in the st.dev. of Ln(A). The sensitivity with respect to the PoD value is reduced but is still negative.

The annual fatigue damage in a given period will vary in a long term perspective. This fact may have large effect on the estimated time to next inspection. A detailed knowledge of the operational history and the As-Is condition for the different time periods e.g. the coating condition, cathodic protection system and corrosion, is required.

The training emphasizes the importance of selecting estimates of PoD, st.dev. of Ln(A) and the ratio between annual accumulated fatigue damage in the period before and after the event of "No Find" or "Detection and Repair" of a fatigue crack on the conservative side. It is mandatory that the user of the databases of PIA analyses know how to select values on the safe side. Hence, the users need to have knowledge of the degree of sensitivity in the scheduling for the selection of PoD, st.dev. Ln(A) and effective service time to the events conditioned on in the PIA analyses.

The theoretical analyses and the quantitative approach by PIA analyses have its limitation in use and application areas. If the quality in the input data is not properly selected and the

analyses are not skillfully carried out, a qualitative approach by experienced engineers, may be preferably used.

3.2 From Craftsman Tradition to Quantitative Analyses

The examples in Section 2.4 and comments in Section 2.5 highlight the importance of the following uncertainty elements:

1) The quality of the calculated characteristic fatigue load effects
   a. The calculated fatigue lives estimates based on the fatigue analyses methodology assuming "normal" quality in fabrication quality and local design according to fabrication drawings used as the basis for the analyses model
   b. The knowledge of as-is condition related to detailed design and fabrication quality
2) The quality of the inspections done
3) The quality of the documentation of the inspection, repair and modification history

The uncertainty elements 1b), 2) and 3 are related to the knowledge accumulated by the engineer responsible for the long term detailed follow-up activity of the unit as well as the site engineers during construction, repairs and modifications. The uncertainty element 1a) is related to the knowledge of the structural analyses engineer, and guidelines for this classification are described in detail in [6]. When the fatigue management is changed from a qualitative to a quantitative approach where the craftsman tradition and engineering judgements by an experienced field engineers are substituted by analyses results obtained by engineers often without practical field experience, the main uncertainty elements considered, are limited to element 1a).

The change from a follow-up regime based on craftsman tradition and experienced engineers' judgement to a quantitative structural integrity management methodology by introduction of PIA require special attention to include the uncertainty elements 1b), 2) and 3 given above. Based om experience and engineering judgement it has been introduced an addition of value 0.2 for the st.dev of Ln(A) if studies on the uncertainty elements 1b), 2) and 3) not have been performed. This uncertainty measure is a selected a priori adjustment based on observations and engineering judgement. Systematic comparisons of observed and predicted occurrence of fatigue cracks or other research activities are required to refine the uncertainty measure i.e. activity 2 in Figure 6.

Ref. [6] and [8] recommend PoD values for MPI and EC inspections. The value in [8] of mean PoD of 1.95mm for below-water inspection is significantly higher than the values given in [6] with a mean value of 1.16mm for below-water inspection. The PoD values in [6] are extracted from a large number of sources and the value from [8] is from one study of field data. However, experience shows a high degree of variation in quality between different inspection teams.

By using a high-quality inspection team and performing independent re-inspections, a mean PoD value as low as 0.25mm could be relevant for burr-grinded and profiled details. This value is significantly lower than the lowest value of 0.4mm in [6]. Hence, a relevant range of the mean PoD values is 0.25 to 2.0mm. All PoD values are related to crack depth even if it is the crack length that is observed. Further studies are required for justification of the selected PoD values and model.

An ideal inspection planning, neglecting that there can be errors in the reporting of experience data, is insufficient. Efforts to improve the inspection process itself are, hence important.

3.3 Comments for the structural integrity management process

The collected data from service activities of inspection, repair, modifications as well as improvement maintenance are an important source of information for the event "fatigue crack growth" [24], [41]. Observations of crack growth or no crack present, provide information for validating the prediction of fatigue crack growth potential, [38].

Traditionally the data from the inspection process have been used for verification of the assumption of the design analyses. Upon detection of structural failure, the default activity is to re-establish the initial design. To implement improvements within the class regime will also require approval of new class drawings. After the re-assurance and re-certification activities were completed, the inspection results were considered historical and of no future use.

To collect experience data and use them for assessment of the structural condition required a new mind set. We may use the term "traditional maintenance management" versus "integrity management". (Traditional maintenance management is used as a term when the process of improvement maintenance is neglected.) The old approach focused on maintaining the conditions established by design, in contradiction to the new approach of integrity management that includes the in-service observations for the assessment of the failure potential independent of the historical design assessments. The integrity management process focuses on the potential need for improvement of a given design to fulfill its function. To support this activity we rely on the quality of the collected experience data.

3.4 Comments on the experience data for semi-submersibles

If the findings for jacket (Section 1.2 and 1.3) are compared with those for semi-submersibles it is found that conclusion a) of over-prediction, is not valid for the semi-submersibles. For the semi-submersibles, there may be an under-prediction of the general fatigue crack growth potential, Table 4 shows 2.8 to 4.2% of the inspections to detect a fatigue crack. Our inspection scheduling methodology expects between 3 and 15% of the scheduled inspections to detect a fatigue crack. However, normally there are more inspections done than the theoretical methods are scheduling. If the average detection rate of propagating crack exceeds 3% of the scheduled inspections, it might indicate an under-prediction of the number of propagating cracks. Then As-Is inspections should be used to identify possible gross errors in fabrication and include the information of actual as-is condition in the prediction of the fatigue crack growth potential. A large per cent of detected fatigue cracks in scheduled inspections might

also indicate shortcomings in the As-Is inspections. At the same time, a low detection frequency in the scheduled inspections, might indicate a need to improve the inspection process. Based on experience with semi-submersible, it is expected that 1 to 5% of the scheduled inspections will report detection of fabrication defects. For randomly selected inspections in area not focused in the fatigue assessment process, the crack detection rate might be as high as 5%.

Conclusion b) that between 10-40% of the observed fatigue cracks developing are detected on hot spot or in service period, which the theoretical analyses would not have scheduled for inspection, seems to be much lower for semi-submersibles. The main reason is the larger extent of re-inspections and the general improved correlation between predictions and observations for the second inspection event.

Moreover, it is believed that conclusions c) and d) in Section 1.2 from the jacket validation study are also valid for semi-submersibles. However, the curve denoted "First Inspection" in Figure 1a seems to be steeper in addition to an increase in frequency of crack occurrence. The curves for the events "Second inspection" for semi-submersibles are expected to be similar to the curve "Second inspection" from the jacket validation study.

4 CONCLUDING REMARKS ON LESSONS LEARNED

The use of probabilistic fracture mechanics models and systematically collected and organized experience data can ensure reliable estimate of the fatigue crack growth potential. Refined deterministic fatigue analyses in combination with probabilistic fatigue analyses need to include uncertainty estimates based on the quality of the in-service inspection process, repair and construction process as well as quality of the as-built structures. When the lifecycle follow up regime includes all of these elements (i.e. a most advanced follow-up regime), it is possible to secure a low potential for fatigue failure and, hence, secure the operational regularity. This performance can even be achieved for structures experience a very high fatigue loading and long service time, i.e. with a characteristic fatigue life estimate between 10 and 15 years and service life of 30 years. As experience data are collected, the estimates of the fatigue crack growth potential can be made increasingly precise and an effective scope for improvement maintenance can be defined.

The majority of observed fatigue cracks are related to poor local design and workmanship. There is insufficient focus on the details in the design, construction and follow-up phase. When cracks are detected the shortcomings are discovered and a scope for improvement maintenance is implemented and the potential for re-occurrence of fatigue cracks are low. Hence, a significant part of the fatigue crack growth is related to the first 5 to 10 years of operation after the steel have been installed. The fatigue failure rate reaches its peak point in this period as long as the unit is properly maintained.

The extent of scope for improvement maintenance will increase with the time in-service as a consequence of the nature of fatigue crack growth. However, as long as the fatigue crack growth potential is properly managed for a structure with a long service life, the fatigue failure probability is kept at a low level, which ensures operational regularity.

The experience data accumulated clearly show a more frequent presence of shortcoming in local design and fabrication quality than considered in fatigue management solemnly based on traditional fatigue analyses.

Moreover, there might be an uncertainty in the methodology that is used to estimate the fatigue damage. For instance fatigue design analyses for mobile offshore drilling units are often based on generic (i.e. worldwide sea scatter diagram) rather than a site and operation specific one, and implies a bias. Another uncertainty could be introduced in the choice of resistance curves depending on the effect of corrosion. Yet another uncertainty is associated with the initiation time for welds improved by grinding.

It should be noted that assumptions that apparently are conservative in fatigue analyses, do not necessarily yield conservative decisions regarding inspection planning in view of inspections with the result "No Find".

The probabilistic fracture mechanics crack growth analyses that make it possible to include the information from in-service observations and fatigue analyses are extremely sensitive w.r.t. input data, and might yield misleading results because of improper data and numerical challenges in connection with complex inspection and repair histories. To ensure reasonably conservative results, establishing input data and carrying out the analyses, should be made by specialists that provide databases of analyses results. These databases should be subjected to quality assurance before they are released for use by the inspection engineers in their scheduling of in-service follow-up activities. Hence, it is important that a SIM team with the proper competence is established.

Our conclusions are based on experiences and collected observation data for mobile semi-submersible platforms. So far only limited quantitative comparisons between observations and predictions in the same manner as made in the validation study for jackets, have been made. Such a validation study for mobile units is more challenging due to a larger number of different hot spot types, frequent structural modifications and changes of location and heading of the unit. At the same time, there is a large number of re-inspections and follow-up inspections after repair and weld improvements. The extent and quality of collected data hold a high promise of obtaining very useful and valuable results from such a study, and is recommended to be carried out.

Generally, the available guidelines for probabilistic assessment of the fatigue crack growth are recommended to be extended. Experiences have clearly shown that a follow up regime at level 2 as defined in Table 6 or establishing of a probabilistic forecast model without the activity 2 of implementation, verification and validation as presented in Figure 6 the probabilistic approach may fail to represent an improvement related to the qualitative approach by experienced engineers.

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[41] Gudmestad, O.T. and Aeran, A. "Guideline for Estimating Remaining Fatigue Life of Ageing Offshore Jacket Structures" OMAE2017 -62059, Trondheim Norway
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APPENDIX A, THE PIA MODEL USED FOR THE JACKET VALIDATION STUDY

A-1 The FM Approach in PIA

PIA is based on the Paris' crack propagation law:

$$\frac{da}{dN} = \begin{cases} C(\Delta K)^m & \text{for } \Delta K > \Delta K_{th} \\ 0 & \text{for } \Delta K \le \Delta K_{th} \end{cases} \qquad \text{(A.1)}$$

where a is crack depth, N is number of cycles, C is crack growth parameter, m is the inverse slope of the SN curve, and ΔK_{th} is a threshold for ΔK = the stress intensity factor (SIF) range given by

$$\Delta K = S\left[Y_{m,plate}\frac{S_m}{S} + Y_{b,plate}\left(1-\frac{S_m}{S}\right)\right]\sqrt{\pi a} \qquad \text{(A.2)}$$

S is stress range and subscripts m and b refer to membrane and bending, respectively. Compliance functions for semi-elliptical surface cracks in flat plates, $Y=Y_{x,plate}(a/t,a/c,c/w,\phi)$, $x = m$ or $x = b$, Ref. 22, are used, where c is one half of the crack length, w is plate width, and the angle ϕ measured from the surface defines a point on the elliptical crack boundary. In PIA, w is set to infinity and ϕ is set to 0.

For applications to tubular joints, $Y_{m,plate}$ and $Y_{b,plate}$ are corrected by a magnification factor M_k in order to account for effects from welds. M_k is taken as

$$M_k = 1 + M_{red}\left[1.0\exp\left(-22.1\frac{a}{t}\right) + 3.0\exp\left(-357.0\frac{a}{t}\right)\right] \qquad \text{(A.3)}$$

M_k as above is based on upper bound (conservative) values, Ref. 30 and 31. The factor reduction factor M_{red} is depending on weld improvements and does not account for load shedding.

Probabilistic Modelling. The probability of any adverse event us fount from

$$p_f = \int_{g(\mathbf{x})<0} f_{\mathbf{x}}(\mathbf{x})\,\mathrm{d}\mathbf{x} \qquad \text{(A.4)}$$

where introducing the *limit state function* $g(\mathbf{x})$ and the joint probability density function $f_{\mathbf{X}}(\cdot)$ for the random vector $\mathbf{X}$ containing the basic input variables. The function $g(\mathbf{x})$ may represent failure, crack detection, etc. Herein, failure is considered to be Through Thickness Crack (TTC).

The reliability index β is defined by $\Phi(-\beta) = p_f$ where $\Phi(\cdot)$ is the standard normal cumulative distribution function. The $g(\mathbf{x})$ function can be obtained by integrating Eq. 1, Ref. 20:

$$g(\mathbf{x}) = \int_{a_0}^{a_1} \frac{\mathrm{d}y}{\left[G(y)Y\sqrt{\pi y}\right]^m} - C\frac{T_1 - T_0}{T_{av}} A^m \Gamma\left(1+\frac{m}{B}\right) \qquad \text{(A.5)}$$

where a_0 and a_1 are initial and final crack depths, respectively. In case of TTC, a_1 = member thickness, whereas in case of crack detection, a_1 = smallest detectable crack size. T_0 and T_1 are the start and end times, respectively. $G(\cdot)$ is defined by

$$G(y) = \frac{\Gamma\left(1+\frac{m}{B};\left(\frac{\Delta K_{th}}{AY\sqrt{\pi y}}\right)^B\right)}{\Gamma\left(1+\frac{m}{B}\right)} \qquad \text{(A.6)}$$

$\Gamma(\cdot)$ and $\Gamma(\cdot\ ;\ \cdot)$ are the Gamma and the complementary incomplete Gamma functions, respectively.

Table A.1 Standard basic input variable values for PIA

Basic input var. name	*Description*	*Distribution*	*Mean or const. Value*	*Coefficient of variation (D indicates standard variation)*
T_{inst}	Installation time	Constant	Input	
A_0	Initial crack size	Exponential	0.11	(D=Mean)
Ln(A)	A is scale parameter in stress range	Normal	Input	0.20 (D)
$1/B$	B is shape parameter in stress range	Normal	Input	0.07 (D)
T_{av}	Average wave period	Normal	6.31	0.05 (D)
Y_1	Correction factor on Y functions, Eq. 3	Lognormal	1.0	0.10
A/c	Aspect ratio for crack	Lognormal	0.15	0.10
$\sigma_{HS}^{(m)}/\ \sigma_{HS}$	Membrane to total stress ratio	Lognormal	0.20	0.10
M_{red}	See Eq. A.4	Constant	1.0	
T	Wall thickness	Constant	Input	
LnC	See Eq. A.2	Normal	-29.84	0.55 (D)
M	See Eq. A.2	Constant	3.1	
ΔK_{th}	See Eq. A.2	Lognormal	100	0.2
A_f	Final crack size	Constant	Input	(D= Mean)
T_{inse}	Time in service (time elapsed = T_{inse}-T_{inst})	Constant	Input	
A_d	Minimum detectable crack size	Exponential	1.2	(D = Mean)
$A_{0,GR}$	A0 after grinding	Exponential	0.05	(D = Mean)
$(a/c)_{GR}$	a/c after grinding	Lognormal	1.0	0.20
$M_{red,GR}$	M_{red} after grinding	Constant	0.5	

10.2 Presentation by Gudfinnur Sigurdsson

DNV·GL

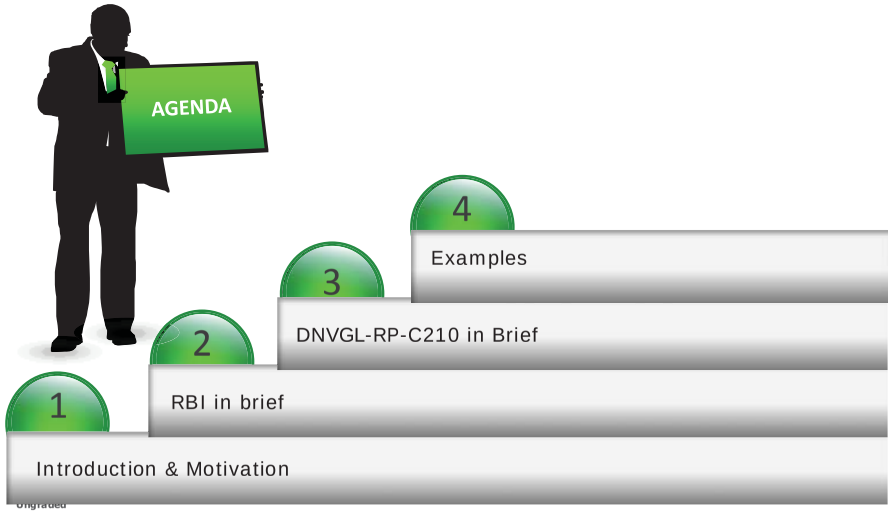
OIL & GAS

Guidelines for Probabilistic Inspection Planning for Fatigue Cracks in Offshore Structures

OSRC 2016 - Session 8: Inspection Planning with Respect to Crack Control

Gudfinnur Sigurdsson & Arne Fjeldstad
16 September 2016

Agenda



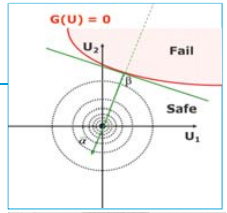
Introduction & Motivation

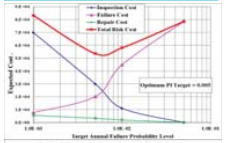
- The number of offshore structures reaching its intended design life is increasing. In many cases there is still oil left on the field ; Thus life extension of offshore platforms is being asked for.
- Inspection of load bearing structures and hulls is a significant cost for operators. Hence there is great demand for more flexible and risk based inspection schemes.
- High focus on operating costs, and operators want to optimise the IMR cost

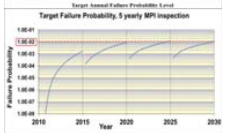
↳ ***There is a need for consistent assessment methodology for structural degradation to maintain acceptable safety level in a cost efficient way***

Introduction & Motivation

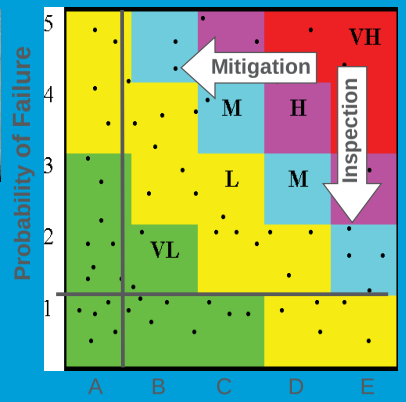
- Probabilistic methods as a tool for documentation of structural reliability has been used for several years, mostly in academia but also by the Oil&Gas industry
- General acceptance of utilising more flexible and risk based inspection schemes are needed in order to balance the inspection effort and the benefits.
- Risk-based inspection planning for fatigue cracks in offshore structures is well establish within many companies.
- RBI is a systematic approach combining theoretical engineering, test results and in-service experiences to maintain an acceptable reliability level.







RBI in Brief



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Purpose of RBI

- Enable control of the risk level over the service life and be able to initiate cost efficient inspections/actions if found required.
- The results of the RBI planning should include a specification of the future inspections in terms of:
  - Where & what to inspect *(which items/areas/tanks)*
  - How to inspect *(GVI, CVI, NDT method, and extent of inspection)*
  - When to inspect *(inspection time)*
  - Directions for actions if defects are found
- Probabilistic models based on the use of SRA methods are required for determination of the Probability of "Failure" of structural components over the service life.

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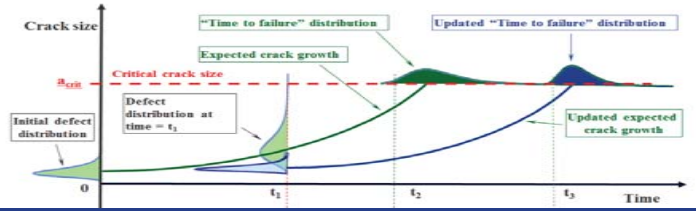
RBI planning

- RBI planning is a "living process":

  Should utilise the **most recent information** regarding the condition of structures, i.e. design, construction, inspection, re-analyses and maintenance of the structures, to optimise future inspections.

- From the inspection results:

  New and better knowledge of degradation rate, loads and capacity gives a basis for an updating of the predicted failure probability and updated time to next inspection may be calculated



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Goal of RBI

- **Increased time** between planned "shutdowns" means more production time
- **Increased reliability** with fewer unplanned "shutdowns" means more production time
- **Optimised inspection frequency** based on environmental, safety and economical risk

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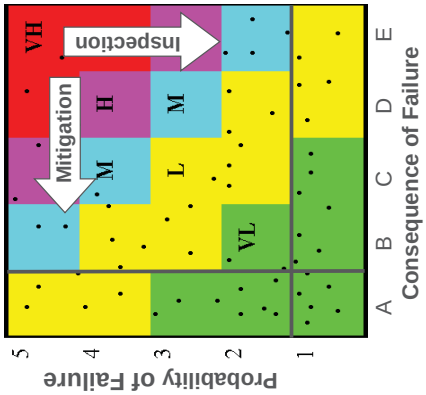
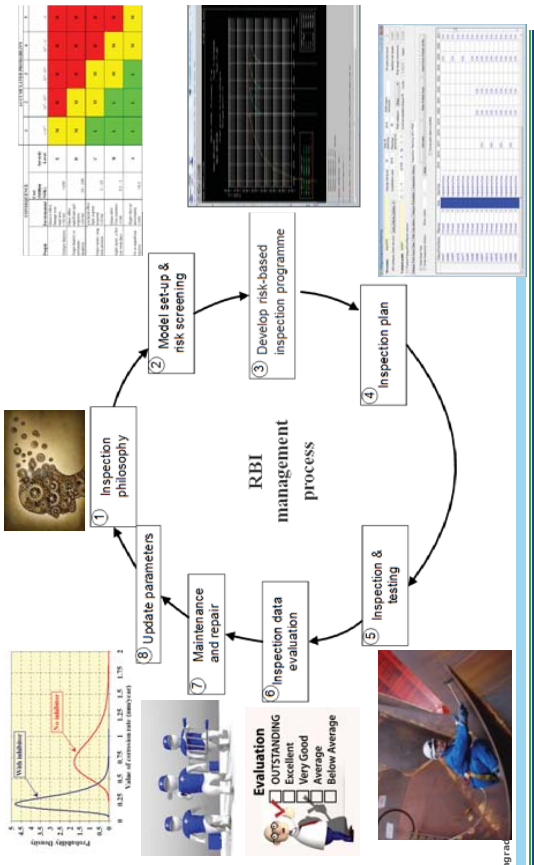
PoF & CoF

Risk (PoF & CoF) vs. Mitigation & Inspections

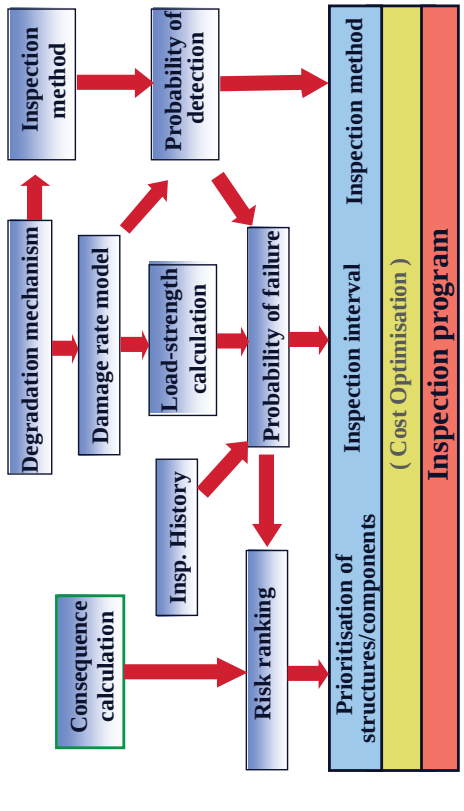
- The PoF increases over time because of time-dependent degradation.
- Inspection increases knowledge of the state of the structure i.e. reduce uncertainties, and may reduce the risk by reducing calculated PoF
- To reduce consequence means instigating mitigating measures



RBI - What is needed?



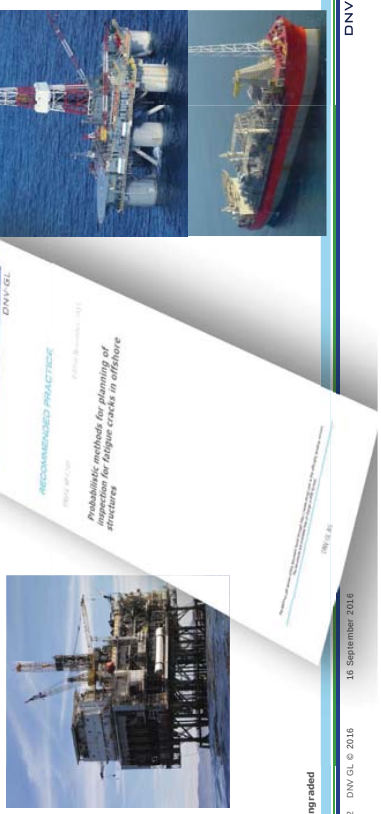
Main principles of RBI



New DNV-GL RP : Joint Industry Project 2011 - 2013

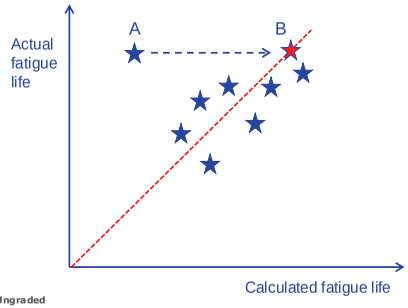
New DNV-GL RP includes sufficient guidance to the user on how to establish inspection planning for fatigue cracks

Main focus on Applicability for Jackets – Semisubmersibles – FPSOs



Fatigue analysis

- Conservative methods and values often used in design fatigue analyses
- In-service inspection to be based on consistent fatigue analysis
- By "consistent" is understood that all potential hot spots are analysed based on similar inherent "conservatism"



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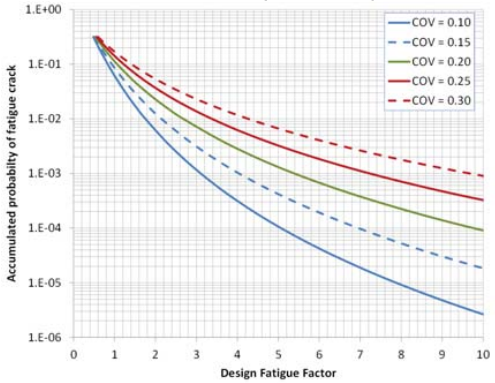
Target accumulated probability of fatigue failure

- Highest probability of fatigue failure that we can accept
- Consequence of fatigue failure
- Uncertainties in the calculated hotspot stress
- Benefit from a good analysis

Effect of uncertainty in hot-spot stress



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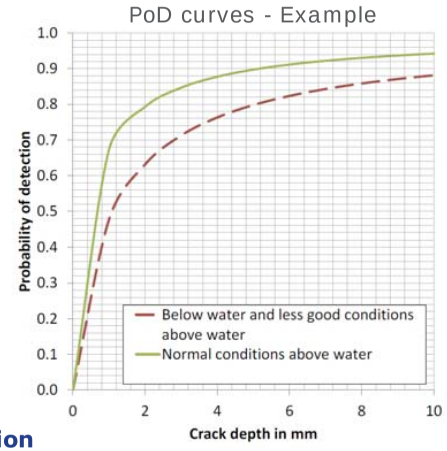
Inspection quality given by PoD curves

- Eddy Current (EC)
- Magnetic Particle Inspection (MPI)
- Ultrasonic Testing (UT)
- Flooded member detection (FMD)
- Leakage detection



Benefit from accurate inspection

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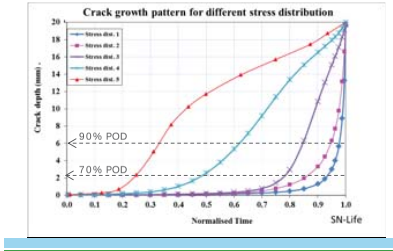
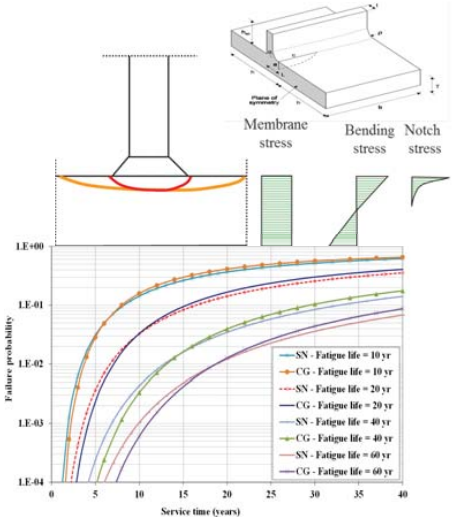
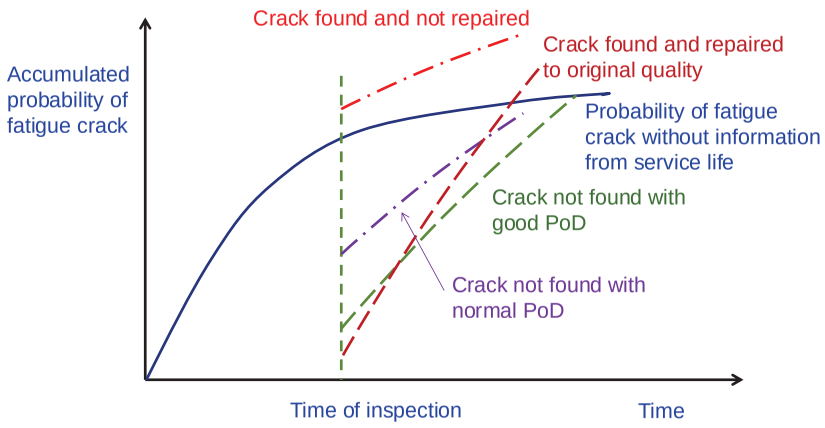
Calibration of fracture mechanics models to S-N data

- Fatigue damage is calculated based on S-N test data
- Fatigue inspections related to crack detection
- Complex link between calculated fatigue damage and crack size
- Calibrated crack growth model
- Initial crack size distribution





Schematic illustration of inspection findings



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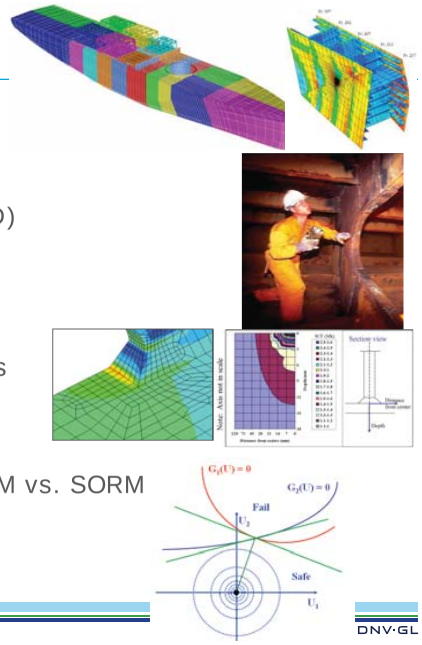
Examples

- Effect of quality of Fatigue analysis on Inspection intervals
- Effect of inspection quality (POD) on Inspection intervals
- Effect of accurate modelling of through thickness local stresses on Inspection intervals
- Effect of solution method : FORM vs. SORM



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Summary DNVGL-RP-C210

- Main document : 125 pages.
- Appendix A Gives advice on fatigue analysis of jackets for purpose of inspection planning: 62 pages.
- Appendix B Gives advice on fatigue analysis of semi-submersibles: 40 pages.
- Appendix C Gives advice on fatigue analysis of floating production ships: 55 pages.
- Appendix D Gives background for recommendations in the main part of the recommended practice.

- DNVGL-RP-C210 was issued November 2015.

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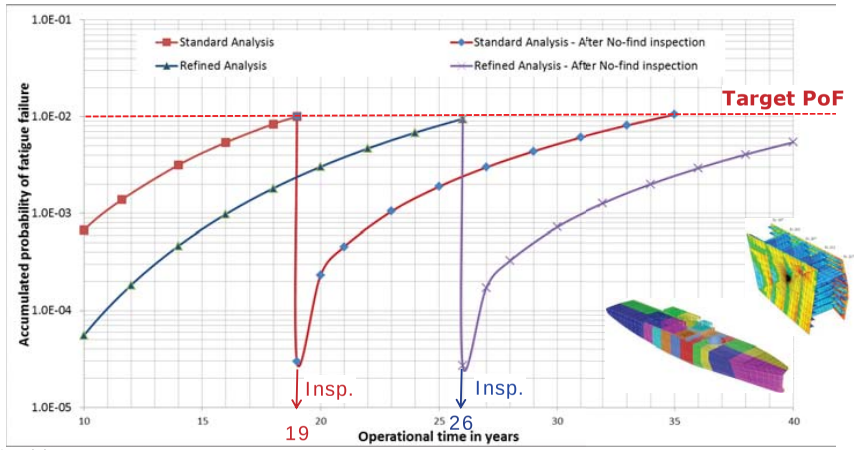
Example

- Effect of quality of Fatigue analysis (Calc. Fatigue life = 60 years)



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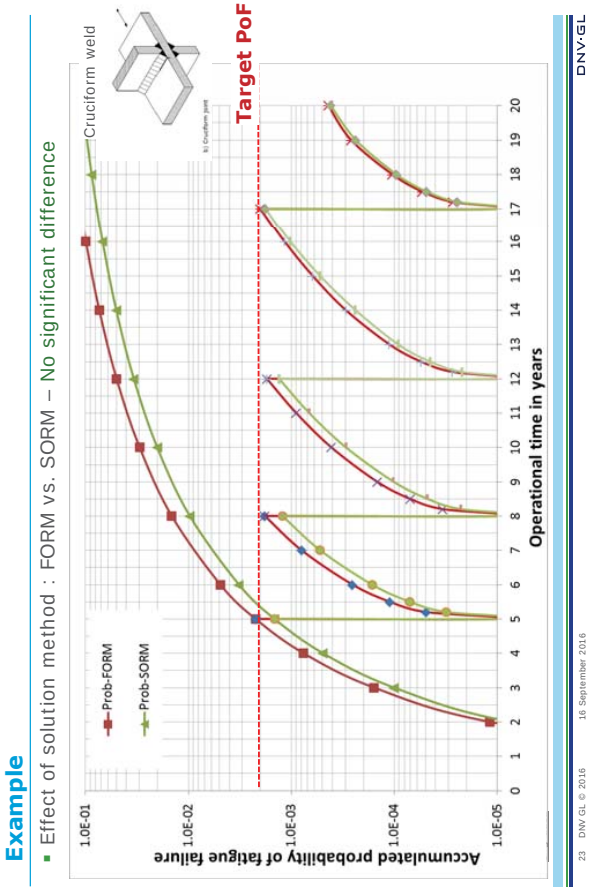
Example
Effect of solution method : FORM vs. SORM – No significant difference
Prob-FORM
Prob-SORM
Cruciform weld
Target PoF
Accumulated probability of fatigue failure
Operational time in years
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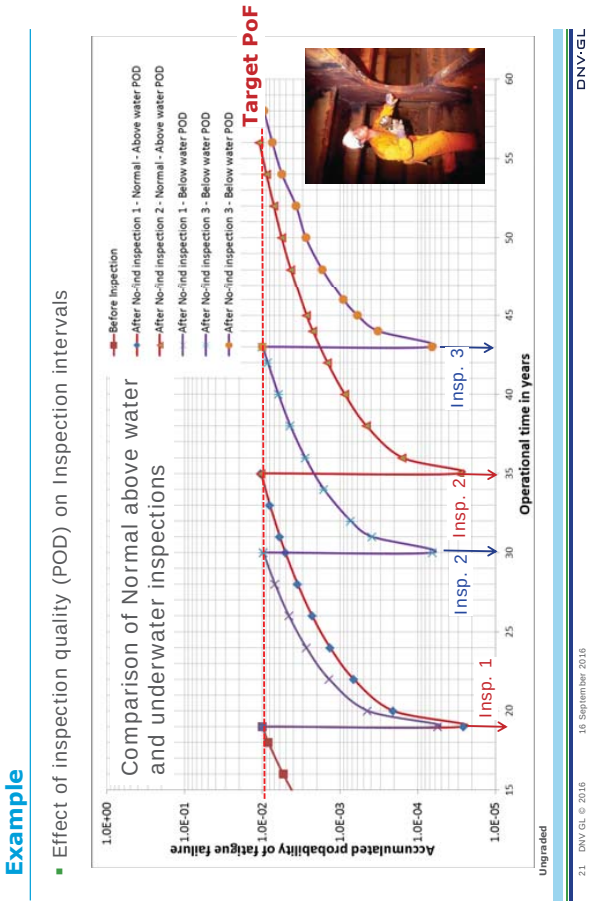
Example
Effect of inspection quality (POD) on Inspection intervals
Comparison of Normal above water and underwater inspections
Before Inspection
After No-ind inspection 1 - Normal - Above water POD
After No-ind inspection 2 - Normal - Above water POD
After No-ind inspection 1 - Below water POD
After No-ind inspection 3 - Below water POD
After No-ind inspection 3 - Below water POD
Target PoF
Insp. 1
Insp. 2
Insp. 2
Insp. 3
Accumulated probability of fatigue failure
Operational time in years
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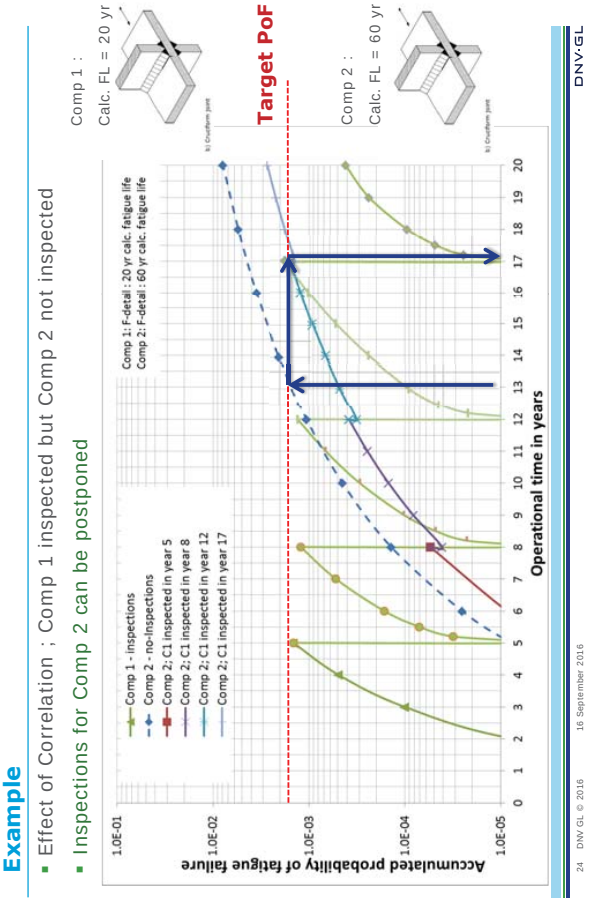
Example
Effect of Correlation ; Comp 1 inspected but Comp 2 not inspected
Inspections for Comp 2 can be postponed
Comp 1 : Calc. FL = 20 yr
Comp 2 : Calc. FL = 60 yr
Comp 1 - inspections
Comp 2 - no-inspections
Comp 2; C1 inspected in year 5
Comp 2; C1 inspected in year 8
Comp 2; C1 inspected in year 12
Comp 2; C1 inspected in year 17
Comp 1; F-detail : 20 yr calc. fatigue life
Comp 2; F-detail : 60 yr calc. fatigue life
Target PoF
Accumulated probability of fatigue failure
Operational time in years
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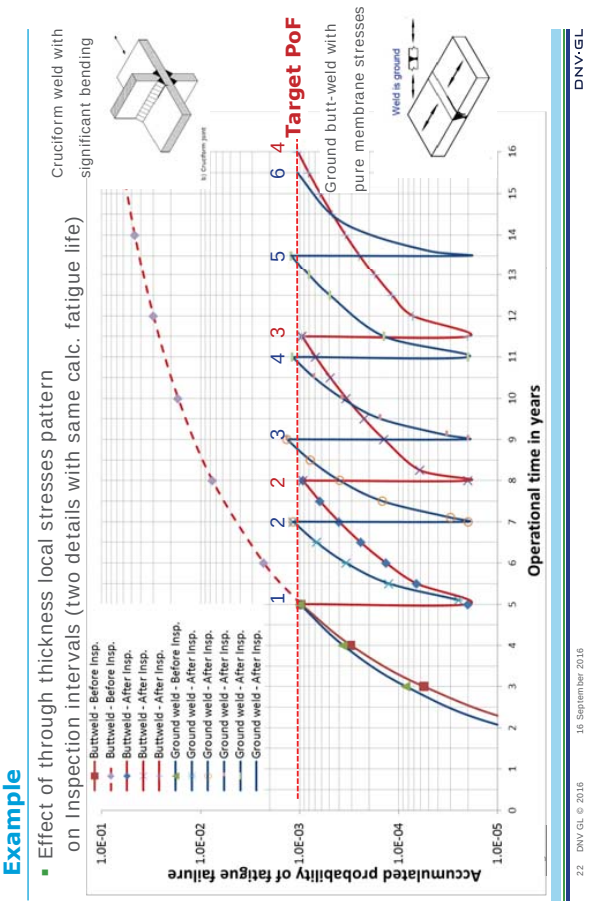
Example
Effect of through thickness local stresses pattern on Inspection intervals (two details with same calc. fatigue life)
Cruciform weld with significant bending
Ground butt-weld with pure membrane stresses
Weld is ground
Target PoF
Accumulated probability of fatigue failure
Operational time in years
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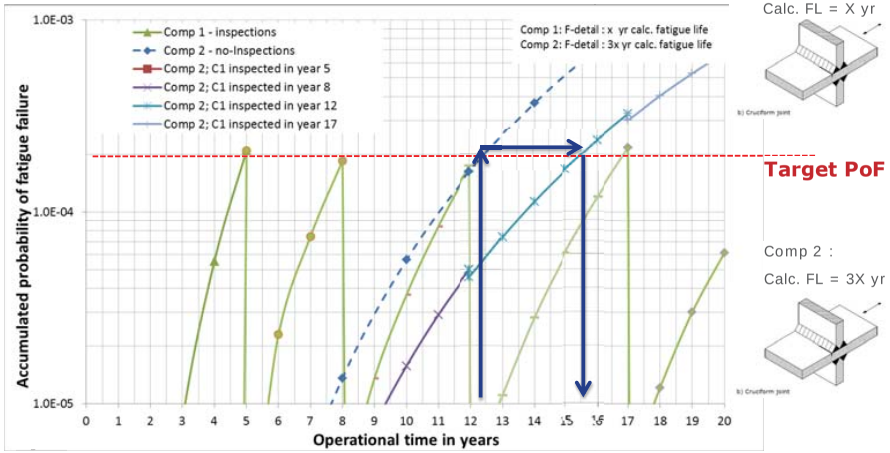
Example

- Effect of Correlation ; Comp 1 inspected but Comp 2 not inspected
- Inspections for Comp 2 can be postponed



Summary

- Optimization of structural inspections using probabilistic or risk based approach may reduce the operational cost significantly
- Important to apply consistent input parameters, have focus on through thickness local stresses pattern, uncertainty modelling and consequence of fatigue cracking (target PoF)
- Industry guidance available to establish sound risk based inspection planning for fatigue cracks (DNVGL-RP-C210)

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Optimization of structural inspections using risk based approach may reduce the operational cost significantly

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SAFER, SMARTER, GREENER

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10.3 Presentation & article by Michel Birades & Laurent Verney

NTNU Norwegian University of Science and Technology

International Association of Oil & Gas Producers

Fatigue Analysis, Lifetime Extension and Inspection Plans

Michel BIRADES
(CSTJF, Pau)

Laurent VERNEY
(M&O, Paris)

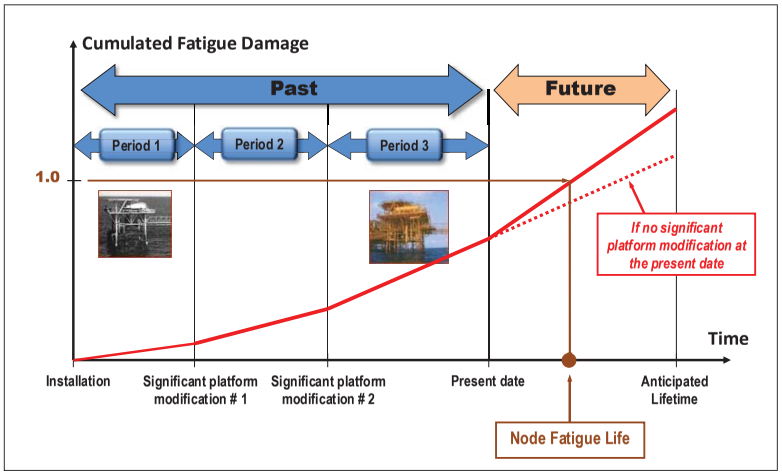
The 3rd Offshore Structural Reliability Conference
OSRC2016
14-16 September, Stavanger, Norway

Introduction

- **Purpose**: Present the global process for lifetime extension according to Total and Bureau Veritas experiences

1. Fatigue analysis:
   - Phasing of the assessment and marine growth evaluation
   - Fatigue analysis overview
2. Lifetime extension and inspection plans
   - Lifetime extension / recertification of offshore structures
   - Inspection plan: Example of TEPNL

Analysis Phases for Fatigue Life Assessment

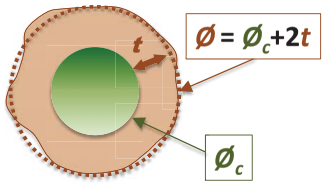


Impact of Marine Growth on Fatigue Analyses

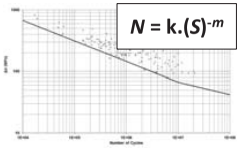
- Marine growth increases apparent diameter, which increases hydrodynamic forces, according to Morison equation :

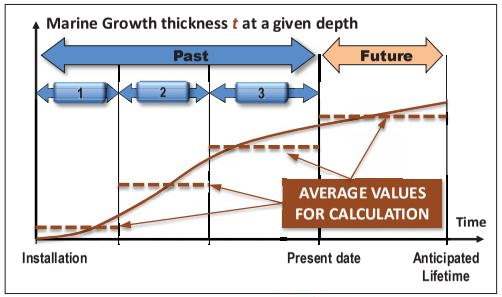
$$F = F_d + F_i$$

$$F = C_d \cdot \frac{1}{2} \rho_w U \cdot |U| \cdot \phi + C_m \cdot \rho_w \pi \frac{\phi^2}{4} \cdot \frac{\partial U}{\partial t}$$



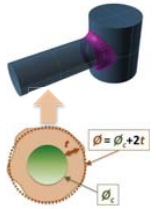
- Marine Growth changes with time
- At each period, an average MG shall be estimated for fatigue calculations
- The simple arithmetic mean value does not reflect the non linearities of the Morison equation and of the S-N curve:





Calculation of average value of Marine Growth

Calculation of the total fatigue damage D_T over Y years on a member of "clean" diameter $Ø_c$ with t_i marine growth thickness at year i



Year	Marine Growth Thickness t	Member Diameter $Ø$	Amplitude stress range S	Number of cycles to failure N	Fatigue Damage D
1	t_1	$Ø_1 = Ø_c + 2t_1$	$S_1 = \lambda.Ø_1^p$	$N_1 = k.(S_1)^{-m}$	$D_1 = n/N_1 = n/k.(\lambda.Ø_1^p)^m$
2	t_2	$Ø_2 = Ø_c + 2t_2$	$S_2 = \lambda.Ø_2^p$	$N_2 = k.(S_2)^{-m}$	$D_2 = n/N_2 = n/k.(\lambda.Ø_2^p)^m$
⋮	⋮	⋮	⋮	⋮	⋮
i	t_i	$Ø_i = Ø_c + 2t_i$	$S_i = \lambda.Ø_i^p$	$N_i = k.(S_i)^{-m}$	$D_i = n/N_i = n/k.(\lambda.Ø_i^p)^m$
⋮	⋮	⋮	⋮	⋮	⋮
Y	t_Y	$Ø_p = Ø_c + 2t_Y$	$S_p = \lambda.Ø_Y^p$	$N_Y = k.(S_Y)^{-m}$	$D_Y = n/N_Y = n/k.(\lambda.Ø_Y^p)^m$
					$D_T = \Sigma D_i$

m = Slope of S-N fatigue curve
p = 1 for drag term of Morison Equation
= 2 for inertia term of Morison Equation

Average value t_{ap} : Defined such as replacing $t_1, t_2, .. t_Y$ by t_{ap} gives the same total damage D_T

Calculation of average value of Marine Growth

Detailed calculations finally give the average MG value t_a:

$$t_{ap} = \frac{\phi_c}{2}\left(\left[\frac{B}{Y}\right]^{1/p.m} - 1\right) \quad \text{with} \quad B = \sum_{i=1}^{Y}\left(1 + 2\frac{t_i}{\phi_c}\right)^{p.m}$$

$$t_a = \frac{t_{a1} + t_{a2}}{2}$$

t_{a1} and t_{a2} rather close

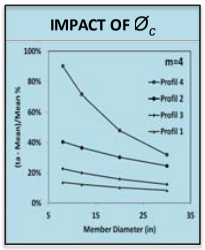
with:

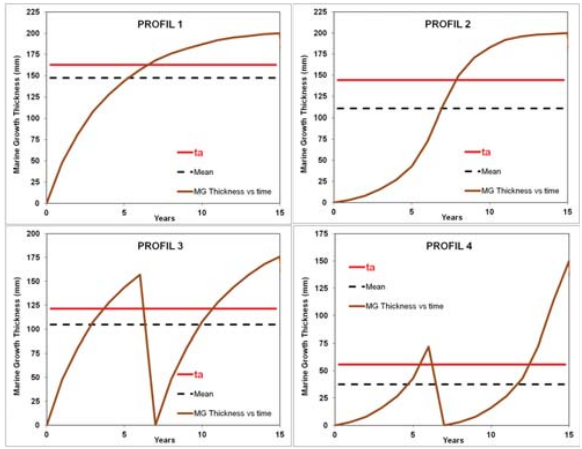
- p: 1 or 2 (Drag and Inertia terms of Morison equation)
- $Ø_c$: Member "clean" diameter
- m: Inverse slope of fatigue S-N Curve
- Y: Number of years
- t_i: marine growth thickness at year i

t_a not sensitive to m

Sensitivity analyses on average MG value t_a

- Selection of 4 typical profiles of MG over time
- Calculation of t_a with $Ø_c$= 20" and m = 4
- t_a is always higher than the arithmetic mean value: *+10 to +50%* for $Ø_c$= 20"





Fatigue analysis overview

- Discussion of typical parameters that may have significant influence on the calculated fatigue lifetime
- Hydrodynamic coefficient and marine growth
  - In fatigue analyses, conservatively the inertia coefficient is set to theoretical value (Cm = 2.0) for both smooth & rough cylinders.
    Note: for in-place analysis:
    Cm = 1.6 for smooth members and 1.2 for rough members
- Deterministic or Spectral approach
  - Before probabilistic approach using a wave spectrum, the environment was described by some deterministic individual waves (i.e. typical annual wave distribution)

Probabilistic approach

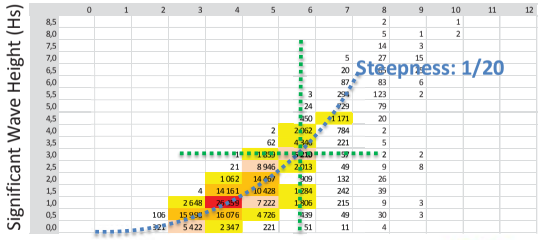
- Spectral approach
  - the spectral approach is a standard computation which is in frequency domain (i.e. linear).
  - its complexity has to be managed by adequate linearization for digesting the environmental (directional) data

Zero Crossing Period (Tz)

Significant Wave Height (Hs)	0	1	2	3	4	5	6	7	8	9	10	11	12
8,5									2		1		
8,0									5	1	2		
7,5									14	3			
7,0								5	27	15			
6,5								20	65	25			
6,0								87	83	6			
5,5							3	294	123	2			
5,0							24	729	79				
4,5							450	1 171	20				
4,0						2	2 062	784	2				
3,5						62	4 346	221	5				
3,0					1	1 859	5 210	57	2	2			
2,5					21	8 946	2 013	49	9	8			
2,0					1 062	14 467	909	132	26				
1,5				4	14 161	10 428	1 284	242	39				
1,0				2 648	26 459	7 222	1 806	215	9	3			
0,5			106	15 998	16 076	4 726	439	49	30	3			
0,0			321	5 422	2 347	221	51	11	4				

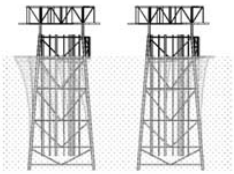
Stress Transfer function (or RAO)

- Linearization
  - The ISO 19902 gives a recipe based on the centre of the fatigue damage scatter diagram.
  - After soil linearization at the centre of the damage, a constant wave steepness is very commonly used for linearization (i.e. establishing the wave height to be used for each wave period).



"Hot spot" approach (or SCF)

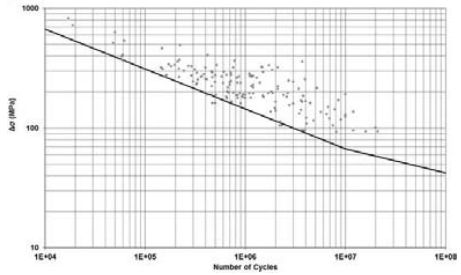
- Stress Ranges
  - The fatigue design of offshore tubular welded joints is based upon the hot spot stress approach (i.e. geometric stress).



  - For each joint, the hot spot stress is then determined using parametric formulas which are called the stress concentration factors (SCFs) and enable to evaluate fatigue stress range at any weld of each joint from the brace stress range.

SN Curves

- The stress range is the parameter which governs the fatigue of welded joints and it is managed by an S-N curve approach.
- From the full-scale testing of tubular joints made in 80s in Europe, a robust design fatigue curve is now available:
  - » design SN Curve with some tests results in air



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Recertification (1/2)

- The matter of Lifetime Extension is to continue to use in satisfactory safety conditions, the offshore platforms that have reached the end of their designed (fatigue) lifetime.
- Bureau Veritas approach for assessing a structural Lifetime Extension was initiated in 1994 based on global scheme which includes the four following steps:
  - Platform Inventory incl. site conditions
  - Re-analysis
  - Survey Program
  - Conclusions

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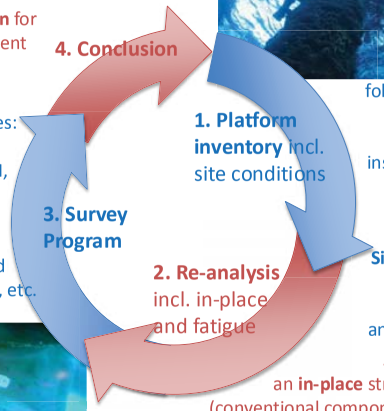
Recertification (2/2)

Decision Phase: The **conclusion** for the lifetime extension assessment is subjected to five years inspection periodicity.

An **inspection program** includes:
i) Annual inspection of overall conditions of the platform: GVI, scour, marine growth, visual inspections of anodes
ii) Structural inspections of the damaged areas: Dents, flooded members, thickness/corrosion, etc.
iii) Specific inspections of appurtenances: Potential readings, riser clamp …
iv) **Critical joints inspections** →



Platform inventory mainly includes the review of initial documents (as-built) with all follow-up reports (weight follow-up topsides & appurtenances arrangements), and periodical inspection reports (Damage with repairs, Corrosion / anodes maintenance and records of marine growth thickness, …).
Site conditions cover mainly the deck clearance reassessment, with possible Metocean loads and Soil resistance reassessment

Structural **re-analysis** includes: an **in-place** structural strength reassessment (conventional component analysis or pushover), and a jacket **fatigue** strength reassessment.

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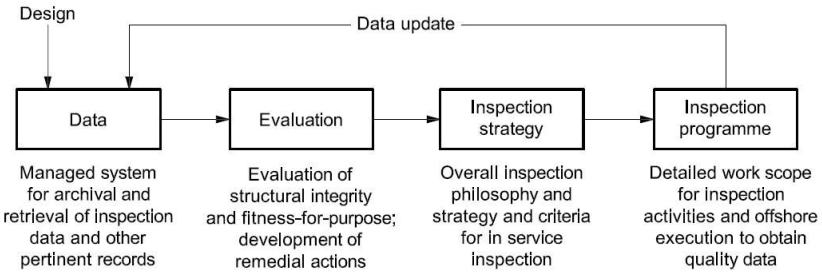
Approach in recent standards

- ISO 19902 and API RP 2SIM have agreed on a Structural Integrity Management (SIM) process which includes the four following phases



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Inspection plan – TEPNL Example (1)

- Inspection of offshore structures in Netherland is regulated by Mining Law, which requires 5 yearly inspections of "critical welds of joints"
- Development of a simple and pragmatic approach, called "**ABCD**", where the assessment of importance of inspecting a weld in a given inspection cycle is a function of:
  - **(A)** Consequence of failure
  - **(B)** Likelihood of failure
  - **(C)** Weld history
  - **(D)** Inspection history

- Overall critical ranking: **TOT** = **(A)**.**(B)**.**(C)** + **(D)**

Inspection plan – TEPNL Example (2)

Welds to be inspected:

1) If ranking score **TOT** is smaller or equal to **5** times the service life, the weld is in the list of welds subject to inspection.
2) Random selection: The number of welds by random selection is equal to 20% of the number of welds obtained in (1) and located below sea level.
3) Minimum number of welds: The total number of welds obtained in (2) must be equal or greater than 5% of the number of primary welds.

TOT = (A).(B).(C) + (D)

Description	(A)
Primary weld	1
Secondary weld	2.5
Tertiary weld	10

Description	(B)
All Welds	FATIGUE LIFE

Description	(C)
No defect found	1.0
Defects ≤ 5mm found & no grinding performed	0.5
Defects > 5mm found & no grinding performed	0.0
Defects found, grinding performed performed & weld found satisfactory	1.0
Defects found, grinding performed & weld found not satisfactory	0.0

Description	(D)
Weld inspected and found to be satisfying	10% x ([A].[B])
All other welds	0.0

Inspection plan – TEPNL Example (3)

Application to a typical TEPNL platform

CALCULATED DAMAGES AND LIFETIMES					
Joint	Brace	Past damage	Future damage	Damage 1994-2034	*Lifetime (Years)*
259	259-295	0,28	2,42	2,7	*26*
119	119-211	0,52	2,14	2,66	*25*
259	215-259	0,27	2,38	2,65	*26*
259	251-259	0,21	1,9	2,11	*28*
251	251-295	0,18	1,58	1,76	*30*
251	251-212	0,15	1,34	1,49	*33*
251	251-259	0,09	0,91	1	*40*
191	191-299	0,2	0,6	0,8	*47*
111	111-291	0,08	0,6	0,68	*51*
115	115-151	0,09	0,5	0,59	*56*
⋮	⋮	⋮	⋮	⋮	⋮

INSPECTION PLAN						
Joint	Brace	(A).(B).(C) + (D)	2018	2023	2028	2033
119	119-211	25	•	•	•	•
191	191-299	47	•	•	•	•
111	111-291	51	•	•	•	•
259	259-295	65	•		•	
259	215-259	65	•		•	
259	251-259	71	•		•	
251	251-295	76	•	•		
251	251-212	82	•	•		•
111	111-115	104	•	•	•	•
211	211-231	105		•		•
251	251-259	110		•		
199	199-219	110	•	•	•	•
115	115-151	141	•	•	•	•
291	291-251	141	•			•
115	115-159	142	•	•	•	•
219	199-219	169			•	
211	211-119	181			•	

• *Above LAT* • *Below LAT*

Conclusion / Summary

- The lifetime extension is a multidisciplinary process which needs different specialists who all are part of the process reliability.
- The data management is the first key for any structural integrity / reliability considerations (refer to the example of Marine Growth).
- The probabilistic fatigue assessment is fed by a number of different parameters (Cm, metocean data, SCF, fatigue curve …) and it also includes implicit linearization that need to be adequately done for keeping the reliability at level.
- For inspection strategy, a semi-quantitative approach based on scores made on consequences, likelihood to fatigue and inspections history, has been deemed reliable for prioritizing underwater inspections over 40 years at TOTAL Netherlands.

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Fatigue Analysis, Lifetime Extension and Inspection Plans

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ABSTRACT: This paper presents a methodology for inspection prioritization by combining failure consequence, fatigue and inspections results. This method has been implemented for a few decades and identifies the joints most critical to the structural integrity in order to inspect the relevant structural components for ensuring a lifetime extension of a jacket structure. This paper presents also the global process for lifetime extension according to Total and Bureau Veritas experiences and discusses evolutions / improvements mainly for marine growth thickness assessment and fatigue analysis performance.

KEY WORDS: Offshore platforms, marine growth, jacket, welded joint, fatigue, inspection plan

1 INTRODUCTION

The offshore industry developments lead to increase demands on existing fixed offshore platforms. The structures are more loaded and may also have reached their initial design life, consequently requiring a lifetime extension.

For existing fixed offshore structures, at least the jacket fatigue analysis has to be updated. This paper will present first the phasing of fatigue analysis over the past and the expected extension with the marine growth thickness assessment for each phase. Then fatigue analysis is summarized with a focus on calculation changes met along the lifetime of the asset.

Finally, the general process for lifetime extension is given with the example of the Total fixed structures in the Dutch waters. The inspection plan approach made by Total with Bureau Veritas is presented for jackets lifetime extensions.

2 ANALYSIS PHASES FOR FATIGUE LIFE ASSESSMENT

An existing fixed offshore platform may be operated within different conditions. The fatigue analysis of the jacket has so to be performed according to the following sequence:

a) Definition of periods over time from *Installation* to *Anticipated lifetime*. A new period is defined when major modification occurs, such as addition of Conductor Pipes, risers, addition of significant weights on the topside (equipment, extension,.. leading to modification of the natural frequency of the platform),…
b) Definition, for each period, of a constant average value of Marine Growth thickness t_a to be taken into account for fatigue calculations. The methodology to calculate this average value is given in section 3.
c) Calculation for each node and for each period of the fatigue damage, in accordance with ISO 19902 [1] and with consideration of node flexibility [2].
d) Calculation for each node of the fatigue damage from *Installation* to *Present Date* (sum of fatigue damages of Past Periods), from *Present date* to *Anticipated lifetime* (Damage during Future Period) and from *Installation* to *Anticipated lifetime* (Total damage = sum of damages of all Periods). Fatigue safety coefficients shall be applied as explained in section 4.
e) Calculation for each node of the Fatigue Life, which shall be calculated as the point in time where the damage fatigue curve crosses Damage = 1.0.

Figure 1 illustrates the methodology, with three past periods and one future period.

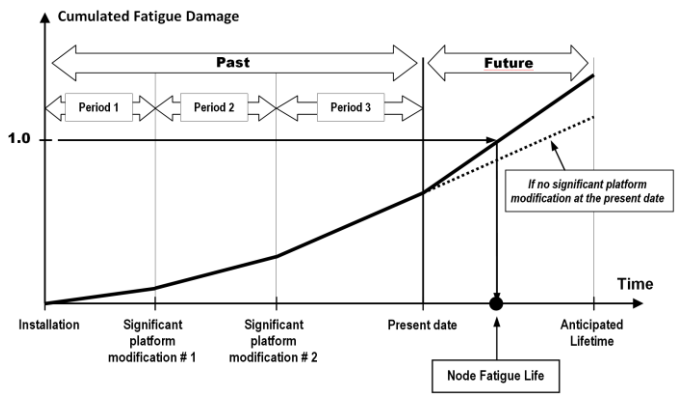


Figure 1. Typical fatigue damages accumulation on a jacket node.

3 AVERAGE VALUE OF MARINE GROWTH THICKNESS

For the fatigue reassessment, the marine growth thickness evolution has to be calculated for all the phases given in the section 2. From *Installation* to *Present Date* ("Past" period), Marine Growth thickness is elaborated from past inspection reports with interpolation between inspections. The Marine Growth thickness from *Present Date* to *Anticipated lifetime* ("Future" period) is extrapolated (Example figures 2 and 3). If, at a given point in time, the curve 'Marine Growth vs. time' is above the maximum allowable value (that should be defined by a sensitivity analysis performed during in-place analyses), the curve is adapted to reflect a cleaning of jacket members at this point in time.

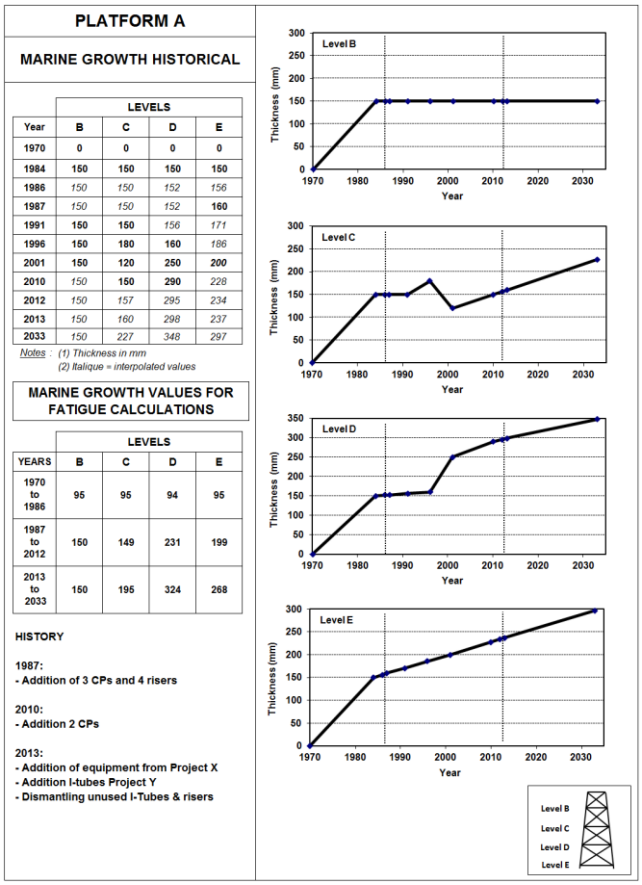
3.1 Impact of Marine Growth on Fatigue Analyses

All elements of the structure (members, conductors, risers, appurtenances) are increased in cross-sectional area by marine growth. The effective element diameter is $Ø = Ø_c + 2t$, where $Ø_c$ is the "clean" outer diameter and t is the marine growth thickness.

Depending on geographical area, the thickness of marine growth can be significant and have a major impact on fatigue life, due to the increased wave and current screens.

If marine growth thickness is different each year, the fatigue damage is also different each year. To avoid performing as many fatigue analyses as the number of years of a given Period, a constant average marine growth thickness t_a is used over the considered Period and calculated as follows.

Figure 2. Example of marine growth on jacket members.



PLATFORM A

MARINE GROWTH HISTORICAL

Year	LEVELS B	C	D	E
1970	0	0	0	0
1984	150	150	150	150
1986	*150*	*150*	*152*	*156*
1987	*150*	*150*	*152*	160
1991	150	150	*156*	*171*
1996	150	180	160	*186*
2001	150	120	250	*200*
2010	*150*	150	290	*228*
2012	*150*	*157*	*295*	*234*
2013	*150*	*160*	*298*	*237*
2033	*150*	*227*	*348*	*297*

Notes : (1) Thickness in mm
(2) Italique = interpolated values

MARINE GROWTH VALUES FOR FATIGUE CALCULATIONS

YEARS	LEVELS B	C	D	E
1970 to 1986	95	95	94	95
1987 to 2012	150	149	231	199
2013 to 2033	150	195	324	268

HISTORY

1987:
- Addition of 3 CPs and 4 risers

2010:
- Addition 2 CPs

2013:
- Addition of equipment from Project X
- Addition I-tubes Project Y
- Dismantling unused I-Tubes & risers

Figure 3. Example of Marine Growth Thickness evolution over time & average value on each of three periods, based on history of platform modifications.

3.2 Calculation of average value of Marine Growth

Table 1 illustrates the calculation of the total fatigue damage D_T over Y years on a member of "clean" diameter $Ø_c$ with t_i marine growth thickness at year i, subject each year i to n cycles of amplitude stress range S_i, during Y years.

As per Morison equation given in ISO 19902 [1], the hydrodynamic force F on a member of diameter $Ø$ is the sum of drag force F_d and inertia force F_i :

$$F = F_d + F_i = C_d \cdot \frac{1}{2} \rho_w U.|U|.\phi + C_m \cdot \rho_w \pi \frac{\phi^2}{4} \cdot \frac{\partial U}{\partial t} \quad (1)$$

The drag force is proportional to $Ø$ and the inertia force is proportional to $Ø^2$

Table 1. Calculation of the total fatigue damage D_T over Y years on a member of "clean" diameter $Ø_c$ with t_i marine growth thickness at year i.

Year	MG Thick. t	Member Diameter $Ø$	Amplitude stress range S	Nbr of cycles to failure N	Fatigue Damage D
1	t_1	$Ø_1 = Ø_c+2t_1$	$S_1 = \lambda.Ø_1^p$	$N_1 = k.(S_1)^{-m}$	$D_1 = n/N_1$
2	t_2	$Ø_2 = Ø_c+2t_2$	$S_2 = \lambda.Ø_2^p$	$N_2 = k.(S_2)^{-m}$	$D_2 = n/N_2$
⋮	⋮	⋮	⋮	⋮	⋮
i	$\boldsymbol{t_i}$	$\boldsymbol{Ø_i = Ø_c+2t_i}$	$\boldsymbol{S_i = \lambda.Ø_i^p}$	$\boldsymbol{N_i = k.(S_i)^{-m}}$	$\boldsymbol{D_i = n/N_i}$
⋮	⋮	⋮	⋮	⋮	⋮
Y	T_Y	$Ø_p = Ø_c+2t_Y$	$S_p = \lambda.Ø_Y^p$	$N_Y = k.(S_Y)^{-m}$	$D_Y = n/N_Y$
					$\boldsymbol{D_T = \Sigma D_i}$

p = 1 for drag term of Morison Equation
= 2 for inertia term of Morison Equation

As a consequence, the amplitude stress range is proportional to the effective diameter $Ø_i$ (= $Ø_c + 2t_i$) for the drag part and to the square of the effective diameter for the inertia part. Therefore at year i:

$$S_i = \lambda \phi_i^p \quad (2)$$

where
- λ is a constant
- p = 1 for drag term of Morison equation
  = 2 for inertia term of Morison equation

According to S-N curve given in ISO 19902 [1], the number of cycles to failure N_i at year i is:

$$N_i = k\, S_i^{-m} \quad (3)$$

where
- k is a constant
- m is the inverse slope of the S-N curve

According to definition given in ISO 19902 [1], the fatigue damage D_i at year i is given by:

$$D_i = \frac{n}{N_i} \quad (4)$$

where

n is number of cycles each year, of amplitude stress range S_i

The total damage D_T on Y years is the sum of the Y annual damages D_i:

$$D_T = \sum_{i=1}^{Y} D_i \tag{5}$$

Considering equations (2) to (5):

$$D_T = \frac{n\lambda^m}{k}\sum_{i=1}^{Y}\left(\phi_i^p\right)^m \tag{6}$$

Considering that $Ø_i = Ø_c + 2t_i$, it finally comes:

$$D_T = \frac{n\lambda^m \phi_c^{p.m}}{k}\sum_{i=1}^{Y}\left(1+2\frac{t_i}{\phi_c}\right)^{p.m} \tag{7}$$

The average marine growth thickness t_{ap} (p=1 for drag term and p=2 for inertia term) is defined as the constant marine growth thickness that generates the same total damage D_T over Y year:

$$D_T = \frac{n\lambda^m \phi_c^{p.m}}{k} Y\left(1+2\frac{t_{ap}}{\phi_c}\right)^{p.m} \tag{8}$$

Considering equations (7) and (8):

$$Y\left(1+2\frac{t_{ap}}{\phi_c}\right)^{p.m} = \sum_{i=1}^{Y}\left(1+2\frac{t_i}{\phi_c}\right)^{p.m} \tag{9}$$

t_{ap} can be extracted from equation (9):

$$t_{ap} = \frac{\phi_c}{2}\left(\left[\frac{B}{Y}\right]^{1/p.m} - 1\right) \tag{10}$$

with

$$B = \sum_{i=1}^{Y}\left(1+2\frac{t_i}{\phi_c}\right)^{p.m} \tag{11}$$

Finally the constant average marine growth thickness t_a to be used over the considered Period of Y years is proposed to be the mean value between t_{a1} and t_{a2}:

$$t_a = \frac{t_{a1}+t_{a2}}{2} \tag{12}$$

This assumption is justified by the sensitivity analyses further down.

It shall be noted that this proposed average marine growth thickness t_a is significantly different from the usual arithmetic mean value defined by:

$$Mean = \frac{1}{Y}\sum_{i=1}^{Y} t_i \tag{13}$$

Furthermore at each level where marine growth has to be taken into account, it is suggested to use the mean value of the diameters of the different members at the concerned level. This will allow avoiding defining a specific t_a for each diameter at the same level.

3.3 Sensitivity Analyses

Marine Growth Profiles

Four Marine Growth thickness profiles have been defined (Cf. figure 4):

- Profile 1: Fast increase of MG thickness from 0 to 200mm during 15 years,
- Profile 2: "S-Curve" increase of MG thickness from 0 to 200mm during 15 years,
- Profile 3: Same as Profile 1, but with cleaning at year 7. After year 7, MG thickness increases as Profile 1 from year 0,
- Profile 4: Same as Profile 2, but with cleaning at year 7. After year 7, MG thickness increases as Profile 1 from year 0.

Tables 2 to 5 summarize the results of the different sensitivity analyses detailed hereafter.

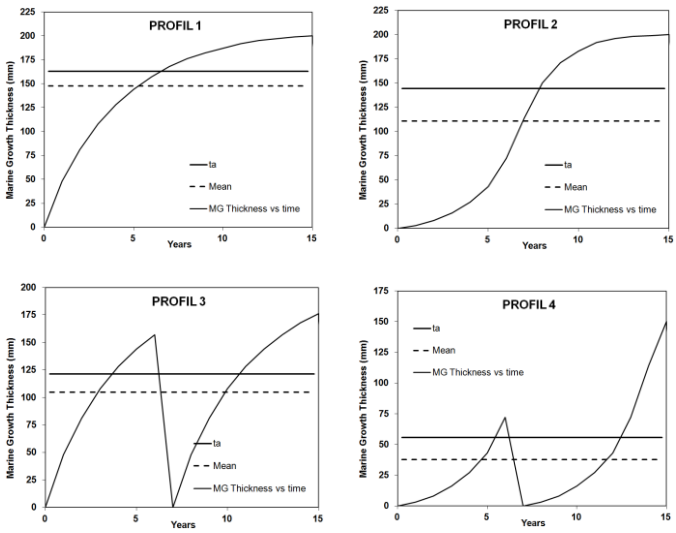


Figure 4. Profiles of Marine Growth thickness over time with *Mean* and t_a (calculated for m=4 and $Ø_c$ =20")

t_a compared to *Mean*

On each curve of figure 4 is indicated the average Marine Growth value t_a, as well as the arithmetic mean value *Mean*. t_a is calculated for a fatigue slope m=4 and a typical member diameter $Ø_c$ =20in.

It appears that t_a is always significantly higher than *Mean*. A more quantified comparison is given on figure 5, which indicates the percentage of increase of t_a vs. *Mean*. The comparison is done for the four selected profiles and for four typical diameters: 8, 12, 20 and 30 in.

t_a is 10% to 90% higher than *Mean*, depending of member diameter and MG Profile. This justifies the use of t_a instead of *Mean* to avoid under conservatism of the fatigue analyses.

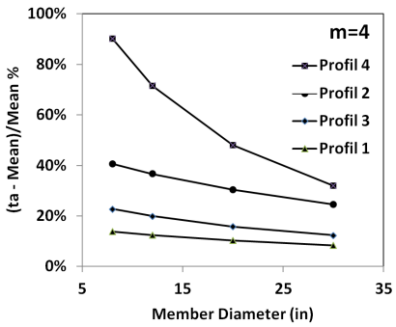


Figure 5. Average MG value t_a increase vs. Mean (%) for the different profiles.

Sensitivity of fatigue slope *m*

Figure 6 compares t_a for different values of the fatigue slope *m*, which usually varies from 3 to 5. The comparison is done for the four different profiles and for the four typical diameters.

It appears that an increase of *m* increases t_a, but the variation is relatively small.

It is therefore suggested to calculate t_a with *m*=4

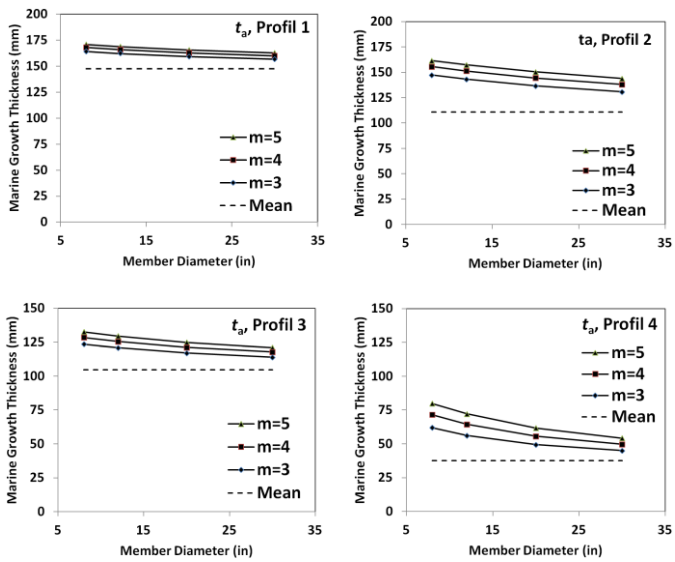


Figure 6. Sensitivity of fatigue slope *m* on average Marine Growth thickness t_a for the different profiles.

Inertia and drag terms t_{a1} & t_{a2} compared to t_a

Figure 7 compares t_{a1} and t_{a2} with t_a. The comparison is done for the four different profiles and for the four typical diameters (with *m*=4 as suggested above).

t_{a1} and t_{a2} appear to be very close to t_a. This justifies the definition of t_a to be the mean value between t_{a1} and t_{a2}. This allows also not to care about which term in Morison equation is predominant.

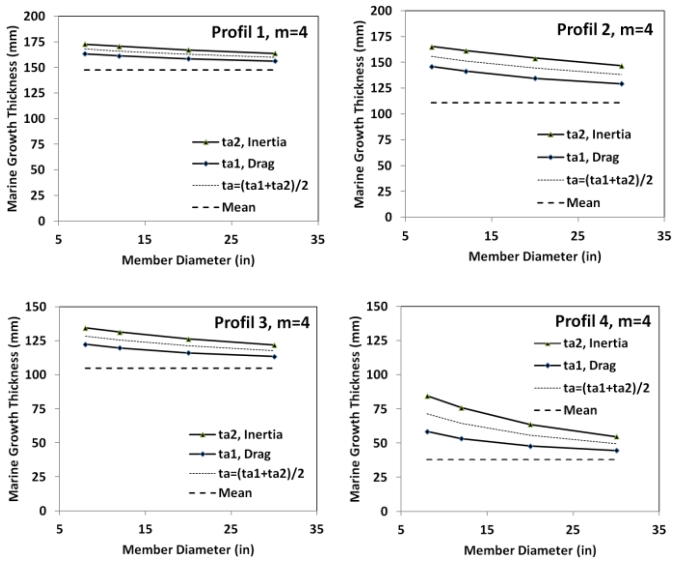


Figure 7. Sensitivity of drag & inertia terms (t_{a1} & t_{a2}) around t_a for the different profiles.

Table 2. Average Marine Growth values - Profile 1.

Diam. (in)	m	$t_{a\ p=1}$	$t_{a\ p=2}$	t_a	t_{Mean}
8	3	159	169	164	148
8	4	163	173	168	148
8	5	166	176	171	148
12	3	158	167	162	148
12	4	161	171	166	148
12	5	164	174	169	148
20	3	155	163	159	148
20	4	158	167	163	148
20	5	161	170	166	148
30	3	154	160	157	148
30	4	156	164	160	148
30	5	159	167	163	148

Table 3. Average Marine Growth values - Profile 2.

Diam. (in)	m	$t_{a\ p=1}$	$t_{a\ p=2}$	t_a	t_{Mean}
8	3	137	158	148	111
8	4	146	165	156	111
8	5	153	170	162	111
12	3	133	153	143	111
12	4	141	161	151	111
12	5	148	167	157	111
20	3	128	146	137	111
20	4	135	154	144	111
20	5	141	160	150	111
30	3	124	138	131	111
30	4	129	147	138	111
30	5	134	153	144	111

Table 4. Average Marine Growth values - Profile 3.

Diam. (in)	m	$t_{a\ p=1}$	$t_{a\ p=2}$	t_a	t_{Mean}
8	3	118	130	124	105
8	4	122	135	129	105
8	5	126	139	132	105
12	3	115	126	121	105
12	4	120	132	126	105
12	5	123	135	129	105
20	3	113	122	117	105
20	4	116	127	121	105
20	5	119	130	125	105
30	3	111	117	114	105
30	4	113	122	118	105
30	5	116	126	121	105

Table 5. Average Marine Growth values - Profile 4.

Diam. (in)	m	$t_{a\ p=1}$	$t_{a\ p=2}$	t_a	t_{Mean}
8	3	51	73	62	38
8	4	59	85	72	38
8	5	66	94	80	38
12	3	48	65	56	38
12	4	53	76	65	38
12	5	59	85	72	38
20	3	44	55	49	38
20	4	48	64	56	38
20	5	52	72	62	38
30	3	42	48	45	38
30	4	45	55	50	38
30	5	47	61	54	38

4 FATIGUE ANALYSIS OVERVIEW

The fatigue analysis is a process which implies a number of different parameters depending on the selected approach. Typical parameters that may have significant influence on the calculated fatigue lifetime of the jacket joints are presented in the seven following subsections.

4.1 Environmental data and wave theory

Fatigue is an accumulation of damage caused by the repeated application of time-varying stresses. For a jacket structure, it results from the environment and more precisely from the waves. So the long-term joint distribution of the significant wave height (Hs) with its representative wave period (Tz or Tp) is needed. The most frequent representation of the long-term wave environment is now the wave scatter diagram, and usually a two-dimensional wave scatter diagram gives the probability density p(Hs,Tz). For existing structures, the initial fatigue assessment may have been done with annual distributions which assume that the conditions during a typical year are repeating themselves each year during the service life (see deterministic fatigue assessment at section 4.2).

The ISO 19902 [1] recommends the use of individual periodic waves for fatigue analysis but several periodic wave theories can be used to predict the kinematics of regular waves. The linear (or Airy) theory is the basic periodic wave theory and is frequently used for fatigue analysis where linearization is necessary (see section 4.5), moreover for computing the Morison equation (see equation 1) a non-linear periodic wave theory may be used as well (see section 4.3).

4.2 Deterministic or Spectral approach

Before probabilistic approach using a wave spectrum, the environment was described by some deterministic individual waves which are periodic (regular) waves with a particular height, period and direction, and an associated number of occurrences. This approach with typical annual wave distribution (in cases without wave scatter diagram, see section 4.1) cannot by nature produce a very realistic representation of the actual sea. This approach is often run through quasi-static analyses (i.e. static analyses where wave loads are magnified with a Dynamic Amplification factor computed analytically) which generally lead to conservative fatigue checks of fixed offshore structures.

To account for the random nature and the frequency content of real waves, the spectral approach is recommended. But considering 8 directions with at least 15 waves per direction and to be computed within a minimum of 12 steps (phase) for having stress range, this approach has been prohibitive for a long time, especially for deep / heavy platforms which need to be analyzed dynamically. Now the spectral approach is a standard computation for all engineering companies, but it remains in frequency domain (i.e. linear) and its complexity has to be managed by adequate linearization (see section 4.5).

For accounting for non-linearity, the time-domain analysis method could be an alternative, but this is almost never used for jacket fatigue assessment, as it is for the floating units.

4.3 Hydrodynamic coefficient and marine growth

For fatigue analyses, most of the waves are lower than the design waves generated during the in-place analysis (1 year extreme waves). Their Keulegan-Carpenter numbers are much lower and the associated value of the inertia coefficient is expected to be larger than those used for in-place analysis ($C_m = 1.6$ for smooth members and 1.2 for rough members). Conservatively the inertia coefficient is set to the theoretical value ($C_m = 2.0$) for both smooth and rough cylinders.

For design waves, several experimentations have been performed. As an example the OTC 13193 [4] with in-place hydrodynamic coefficient showed the same extreme response as the measurements made on Ekofisk jackets in Norwegian waters, but no conclusion is given for fatigue waves. In the end, the conservatism due to C_m in the fatigue wave loading is generally not well identified and remains also to be linked to marine growth (see section 3).

4.4 Structural modeling and joint flexibility

The spectral and deterministic fatigue analyses methods (see section 4.2) are based on stress ranges which are computed from individual periodic waves. The resulting stress cycle at each joint consists of contributions from axial and bending stresses within braces.

For fatigue analysis, the global 3D frame model used for in-place analysis is reused. Basically the members are connected at nodes where the continuous member is called the chord, and member(s) connected to it are called the braces. The assembly of chord and braces is the joint where the tubular connection welds may be prone to fatigue.

New jacket structures are usually computed with the assumption that the joints are rigid. However, it is known that the flexibility of tubular joints may help for increasing fatigue lifetime, and since 1993, Buitrago et al. [2] have published some equations for linear elastic flexibility and stiffness of tubular joints. These Buitrago's formulations are now available in most structural analysis tools and frequently used for fatigue assessment of existing jackets.

4.5 Linearization and Stress Transfer function (or RAO)

The deterministic method used through quasi-static analyses (see section 4.2) does not require linearization, but the spectral method needs linearization, as this frequency approach is applicable to linear systems. The global analysis which produces the nominal stresses, needs so the linearization of hydrodynamic drag actions (the squared drag term in Morison's equation), the inundation effects (around the still water level as the wave surface moves from trough to crest), and the soil-structure interaction.

This linearization is implicitly made through the wave selection for the global response computation; therefore the structural analyst has to pay attention to this phase when selecting the wave heights for transfer functions.

The ISO 19902 [1] gives now a recipe based on the center of the fatigue damage scatter diagram. The wave heights used for the transfer functions must be selected in a suitable manner for having an appropriate level of non-linear wave loading (see wave theory at section 4.1). After a soil linearization at the center of the fatigue damage scatter diagram, constant wave steepness (ratio of wave height to wave length, usually in the steepness ranges from 1:15 to 1:25) is very commonly used for establishing the wave height to be used for each wave frequency. As this constant wave steepness gives unrealistically large wave heights at small wave frequencies, a maximum height equal to the maximum height in the scatter diagram is also commonly implemented. Additionally a wave calibration process could be made for confirming the wave steepness value which matches the spectrally calculated response to a deterministically calculated response for all sea-states directions.

Further to this linearization, under a harmonic excitation at a particular frequency (input), the response of the structure (output) is also harmonic with the same frequency with a phase shift between input and output. The transfer function is defined as the ratio of the amplitude of the output to the amplitude of the input. This ratio is also known as the response amplitude operator (RAO). Finally the checks of the transfer function is made by plotting the total base shear force transfer function for few wave directions in order to ensure good definition of peaks and valleys, especially near the platform natural frequencies where frequency adjustment may be needed for the good shape of all transfer functions.

Using the transfer functions (or RAOs) the wave spectra are computed in order to obtain the corresponding response spectrum for each sea state. A statistic process is finally performed by integrating this response spectrum for having the mean square response, from which the RMS stresses could be computed with its moment. These parameters define the probability distribution function for the stress response range (see section 4.6). The Rayleigh distribution is the more appropriate distribution function for narrow-banded processes and is commonly used for offshore fixed structure response.

4.6 SCF and Fatigue Stress Range

Fatigue design of offshore tubular welded joints is based upon the hot spot stress methodology which used the S-N curve approach (see section 4.7) and considers the design stress to be the geometric or "hot spot" stress.

Offshore jacket models (see section 4.4) are carried out using beam models thus nominal stresses are calculated at member ends. For each tubular joint, the hot spot stress is then determined using parametric formulas providing the stress concentration factor for the considered loading mode (axial, in plane bending, out of plane bending) and simple joint type (T/Y, K, KT, X). These parameters are called stress concentration factors (SCFs) and enable to evaluate fatigue stress range at any weld of each joint from the brace stress range. The Efthymiou equations are now recommended by most of offshore design codes, if they are used with the influence functions for generalizing the joint classification and with the effect of chord stress / length.

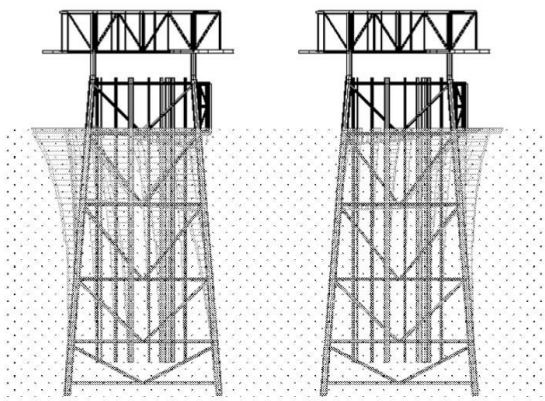


Figure 8. Illustration of fatigue ranges.

Whatever a stress range comes from a probabilistic determination using spectral method or a deterministic analysis using individual wave, it is recognized as the only parameter which governs the fatigue of welded connections and it is managed by an S-N curve approach (see section 4.7).

The "hot spot" stress approach uses the axial stress range with in-plane bending and out-of plane bending stress range and keep them separate. For capturing the influence of constant stress, the 'structural stress' approach presented in [5] shows an improvement with the consideration of the dead weight, like for connections on the jacket legs.

4.7 S-N curve, Miner damage and safety factors

The API RP 2A [6] defines an 'S-N curve' as '*a representation of empirically determined relationships between stress range and number of cycles to failure, including the effects of weld profile and discontinuities at the weld toe*'. This empirical approach results from full-scale testing of tubular joints until fatigue failure in 80s in Europe. These large-scale efforts have significantly increased the amount of available data, and have led to a robust design fatigue curves as given on figure 9, with some tests results.

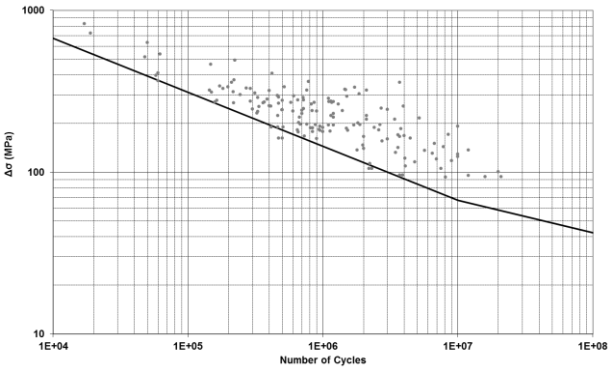


Figure 9. S-N curve
(design curve in air with full-scale test data).

The formulation of the S-N curve is already given in section 3 (see equation 3). The fatigue design S-N curve is confirmed by both ISO 19902 [1] and API RP 2A [6]. It integrates a thickness effect material thickness above 16 mm.

Finally the fatigue assessment is a cumulative damage; according to the linear Miner rule which is already given in section 3 (see equation 5). Both ISO 19902 [1] and API RP 2A [6] have also agreed on the Fatigue Life Safety Factors. The design safety factors are given in the table 6.

Table 6. Fatigue damage design factors.

Failure critical component	Inspectable	Not Inspectable
No	2	5
Yes	5	10

It shall be noted that the values of Table 6 are not appropriate for definition of inspection plan for lifetime extension (For example, their use would lead to inspect some non inspectable component). Therefore the Fatigue Life Safety Factors shall be set to 1.0 when the fatigue damages are used to define the inspection plan (see section 6).

5 LIFETIME EXTENSION

5.1 Approach for recertification

The matter for a Lifetime Extension is to continue to use in satisfactory safety conditions, the offshore platforms that have reached the end of their designed (fatigue) lifetime.

Bureau Veritas approach for assessing a structural Lifetime Extension was initiated in 1994 and was presented at the 2007 OGP workshop [7]. It is based on global scheme which includes the four following steps:

1. Platform Inventory
2. Re-analysis
3. Survey Program
4. Conclusions

The inventory of platform conditions is the review of the reports that consolidate the picture of the platform in its actual situation with an historical background. The inventory shall collect data from the original design, periodical inspection results, site-work reports, changes in loading and/or use and other available documents which have been issued through the life of the platform until its reassessment before the lifetime extension.

The re-analyses shall provide an up-to-date picture of the platform strength and fatigue resistance by including the latest platform design conditions with possible platform updates coming from the Inventory. The purpose of this second review is to confirm the suitability of structures for an extended lifetime, and to provide proper data for later inspections of the platform during its expected lifetime extension period.

A Survey program should be reviewed after the re-analyses, in order to complete the evaluation of conditions of platform, and to correlate its present condition with the finding of fatigue re-analysis. In North Sea, the 5-year structural inspection program covers but not limited:

- Inspection of platform overall conditions (marine growth, cathodic protection, etc.)
- Inspections further to findings in previous campaigns (out-of-straightness, corrosion, flooded members, etc.)
- Inspections of specifics areas (riser clamps, etc.)
- Close inspection and NDT of selected joint welds.

From the results of the three above steps, Bureau Veritas concludes on the structural suitability of the platform for its lifetime extension.

5.2 Approach in recent standards

ISO 19902 [1] and API RP 2SIM [8] have agreed on a Structural Integrity Management (SIM) process which includes 4 phases given on figure 10. This process is similar to the recertification scheme presented at section 5.1.

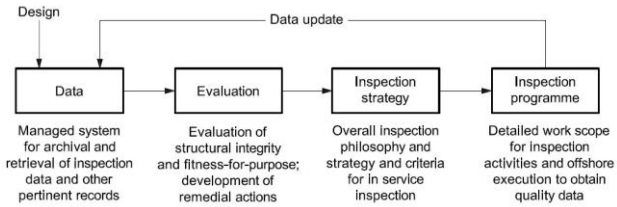


Figure 10. ISO 19902 [1] Phases of SIM cycle

6 INSPECTION PLAN – EXAMPLE OF TEPNL

6.1 General

The inspection of offshore structures of Netherland Total's affiliate (*TEPNL*, ***T**otal **E**xploration **P**roduction **N**etherLand*) is regulated by Mining Law: Article 7.2 section 2G of Mining Regulation of January 2003 requires 5 yearly inspections of "critical welds of joints".

This requirement has been understood as a Level IV survey (as per API RP 2SIM [8]), meaning NDT of the welds of the most critical joints.

Considering that the principal objective of any weld inspection plan is to provide a level of confidence in the condition of each jacket component commensurate with the consequences of failure, a specific approach to this optimization of subsea weld inspection has been developed. This approach (called "ABCD", see further down) is shown as a flowchart in Figure 11.

The flowchart portrays the principal considerations which have a bearing on the inspection program. The assessment of importance of inspecting a weld in a given inspection cycle is a function of:

- Consequence of failure
- Likelihood of failure
- Weld inspection history
- Certifying requirements

Therefore a ranking tree is developed which accommodates all of above. The approach portrayed on the flowchart implies a unique program for each cycle designed on the basis of all available information particularly engineering and inspection results.

After each inspection, new weightings can be assigned as a result. The (revised) ranking table is then used for next inspection. In this way new priorities for inspection of jacket components can be highlighted each cycle.

The report is covering all subsea welds and includes also the above water elevation of the jacket.

The inspection of the pile jacket connection is included in the aerial inspection program.

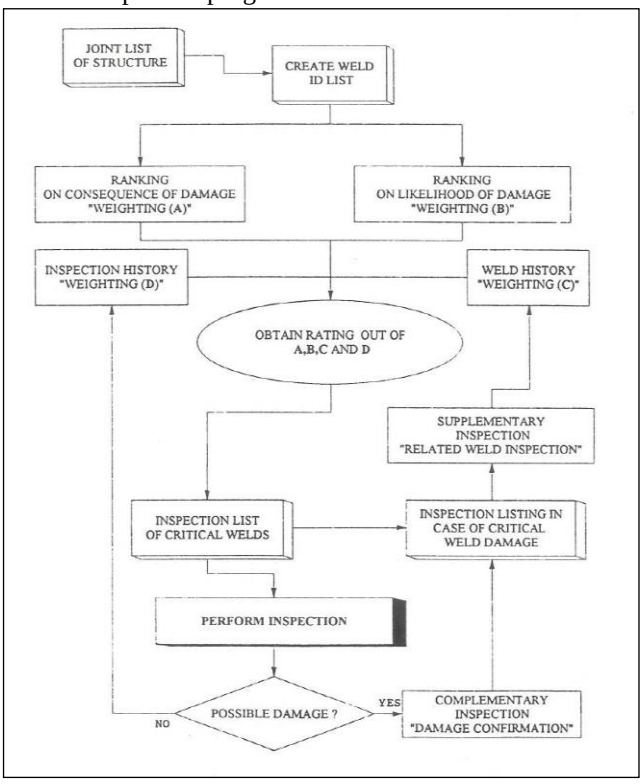


Figure 11. Optimization of subsea weld inspection. Method "ABCD".

6.2 *Critical rating of components*

In order to obtain a ranking table, weighting factors (**A**, **B**, **C** and **D**) are developed. The factors affecting this ranking are explained in the sections here below.

Consequence of failure (A)

Object of this weighting is to classify the structural components of the jacket as primary, secondary or tertiary according to its duty and its importance for the overall stability and integrity of the structure.

- Primary weld includes:
  - welds on either side of braces connected to main legs
  - welds on either side of horizontal members to main legs
  - (welded) riser stubs
- Secondary weld includes:
  - welds of members connecting to members with primary welds
  - conductor framing components not stiffened by welded plates or gussets
- Tertiary weld includes:
  - welds of members connecting to members with secondary welds
  - conductor framing components stiffened by welded plates or gussets
  - all other welds

Weighting scores on these components are according Table 7.

Table 7. Consequence of failure weighting (A).

Description	**(A)**
Primary weld	1
Secondary weld	2.5
Tertiary weld	10

Likelihood of failure (B)

The likelihood of failure is a function of both stress and fatigue. Because weld inspection is focused on detection of weld cracks, fatigue life is the main criterion for the likelihood of failure.

Structures designed to current standards are unlikely to fail under static loading other than from, exceptional loading, inadequate corrosion protection or faulty materials or workmanship.

For an ultimate load failure (member static stress, buckling and punching shear), there is generally little evidence of distress until a local failure has occurred, and this may be detected by a general visual inspection (ROV) in case of exceptional loading incidents.

The likelihood of failure weighting (B) is thus obtained from fatigue life only. However, when it comes to selecting welds with approximately the same ranking, the weld with the highest punching shear ratio will be selected.

$$(\mathrm{B}) = \mathit{FATIGUE\ LIFE} \tag{14}$$

Weld history (C)

Major object to introduce a weld history weighting is to monitor repaired and grinded welds and unfinished inspections. Weighting scores proposed on weld history is according Table 8.

Table 8. Inspection history weighting (C).

Description	(**C**)
No defect found	1.0
Defects ≤ 5mm found & no grinding performed	0.5
Defects > 5mm found & no grinding performed	0.0
Defects found, grinding performed & weld found satisfactory	1.0
Defects found, grinding performed & weld found not satisfactory	0.0

Inspection history (D)

Object to introduce an inspection history weighting is that for two components with otherwise practical the same total weighting, it is preferable to examine in the next cycle the component which was not inspected in the previous cycle. Therefore weighting D is introduced according Table 9.

Table 9. Inspection history weighting (D).

Description	(**D**)
Weld inspected and found to be satisfying	10% x ([A].[B])
All other welds	0.0

Overall critical rating

The proposed formula to calculate the overall critical rating is as follows:

$$\mathrm{TOT} = (A).(B).(C) + (D) \qquad (15)$$

6.3 *Related welds*

If during an inspection defects on a weld with a non-satisfactory result are detected, related welds are to be checked. The related welds are included in the inspection ranking lists based on similar relationships (symmetry and fatigue life) and/or other welds in the same node (e.g.: weld in X or K joints).

6.4 *Quantifying inspection effort*

The number of welds to be selected for every cycle depends on the following factors:

- service life of the jacket
- ranking table score
- size and complexity of the jacket
- certifying requirements

To obtain the number of welds to be selected the following guidelines are used:

1) If ranking score is smaller or equal to 5 times the service life the weld is subject to inspection. The safety factor of 5 covers inspection of secondary welds in case fatigue life is equal or smaller than 2 times service life.
2) Random selection: The number of welds by random selection is equal to 20% of the number of welds obtained in (1). However, since the top elevation (above sea level) is easily accessible, welds at this level may be inspected at each inspection campaign. Thus, the random selection applies only on welds below sea level. And for practical reasons bottom elevation should be avoided if ranking is low.
3) Minimum number of welds: The total number of welds obtained in (2) must be equal or greater than the minimum number of welds required for selection. This minimum is depending on the size of the platform and is 5% of the number of primary welds.

6.5 *Weld inspection operations*

Figure 12 shows a flow chart which explains the weld inspection procedure.

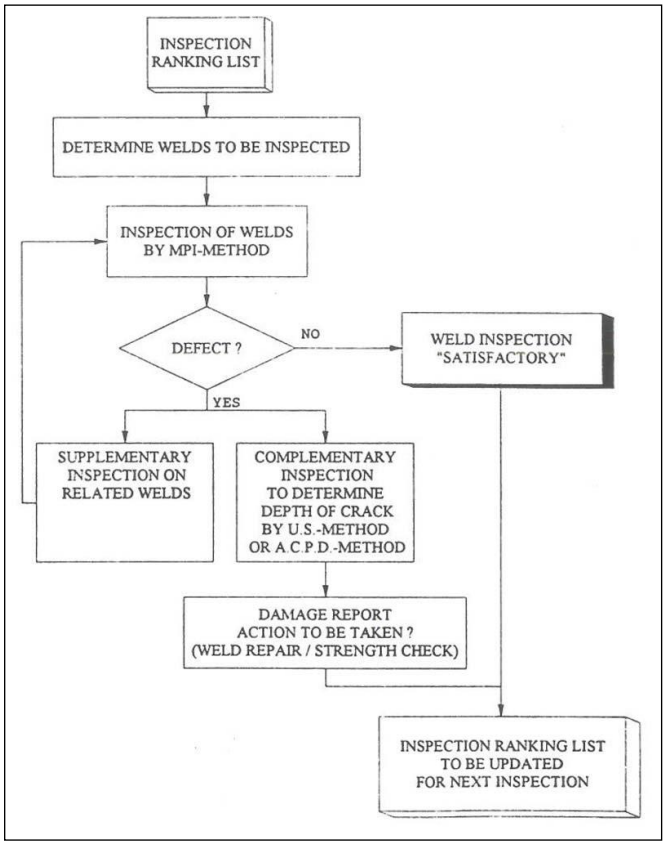


Figure 12. Weld inspection procedure.

6.6 *Example of application "ABCD" inspection plan*

Example of application of "ABCD" methodology is given on a typical TEPNL platform: a 4 legged main-pile jacket, supported by foundation piles driven to 72.00m penetration depth, illustrated on figure 13.

Fatigue damages have been calculated according to the methodologies presented in sections 2 to 4. Installation date is 1994. Anticipated lifetime is 2034 (40 years). Table 10 presents the joints with the highest damages.

The inspection plan elaborated according to "ABCD" methodology is given on Table 11. All joints with ranking (A).(B).(C)+(D) > 200 (5 x 40 years) are indicated. The joints above LAT, easy to inspect, are included every 5 years.

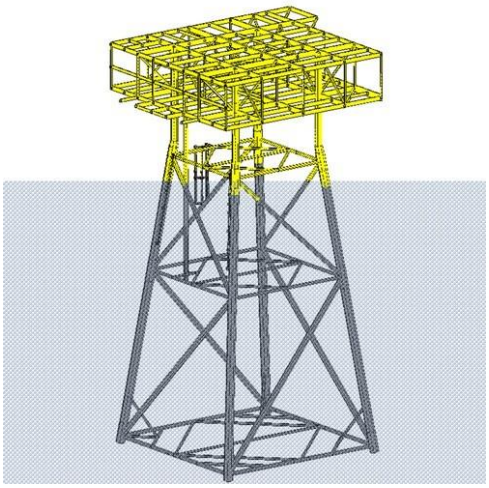


Figure 13. Typical TEPNL platform overview.

Table 10. Calculated damages.

Joint	Brace	Past damage	Future damage	Damage 1994-2034	*Lifetime (Years)*
259	259-295	0,28	2,42	2,70	*26*
119	119-211	0,52	2,14	2,66	*25*
259	215-259	0,27	2,38	2,65	*26*
259	251-259	0,21	1,90	2,11	*28*
251	251-295	0,18	1,58	1,76	*30*
251	251-212	0,15	1,34	1,49	*33*
251	251-259	0,09	0,91	1,00	*40*
191	191-299	0,20	0,60	0,80	*47*
111	111-291	0,08	0,60	0,68	*51*
115	115-151	0,09	0,50	0,59	*56*
⋮	⋮	⋮	⋮	⋮	⋮

Table 11. Inspection Plan.

Joint	Brace	(A).(B).(C) + (D)	2018	2023	2028	2033
119	119-211	25	○	○	○	○
191	191-299	47	○	○	○	○
111	111-291	51	○	○	○	○
259	259-295	65	●		●	
259	215-259	65	●		●	
259	251-259	71	●		●	
251	251-295	76	●	●		
251	251-212	82	●	●		●
111	111-115	104	○	○	○	○
211	211-231	105		●		●
251	251-259	110		●		
199	199-219	110	○	○	○	○
115	115-151	141	○	○	○	○
291	291-251	141	●			●
115	115-159	142	○	○	○	○
219	199-219	169			●	
211	211-119	181			●	

● *Above LAT* ○ *Below LAT*

7 CONCLUSION

The following summary and conclusions are made with regard to Total and Bureau Veritas experiences about the challenges in using aging fixed structures beyond the original design lifetime:

- The lifetime extension is a multidisciplinary process which cannot be seen as a sequence of independent assessments, despite the fact that they generally involve different specialists like subsea inspections expert, metocean specialist, structural lead analyst, etc. who all are part of the process reliability.
- The data management is the first key for any structural integrity / reliability considerations. As an example, the issue of Marine Growth in warm waters needs detailed and repeated measurements for predicting its evolution / cleaning.
- The complexity of the structural evaluation like fatigue has to be well controlled for probabilistic determination using spectral method which is fed by a number of different parameters (metocean, hydrodynamic and structural) and it also includes implicit linearization that need to be adequately done for keeping the reliability at level.
- For inspection strategy, a semi-quantitative approach based on scores made on consequences, likelihood to fatigue and inspections history have been sufficiently reliable for prioritizing underwater inspections during 40 years at Total Netherlands.

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Chapter 11

Session 9: Reliability of Mobile Units

11.1 Presentation by Tore Sildnes

NTNU
Norwegian University of Science and Technology

International Association of Oil & Gas Producers

Operational Experiences and Design Codes for MODUs

Tore Sildnes / DNV GL

The 3rd Offshore Structural Reliability Conference
OSRC2016
14-16 September, Stavanger, Norway

2016 Offshore Structural Reliability Conference

Operational Experiences and Design Codes for MODU

Contents of presentation:

- Regulatory Basis
- Role of Class
- Design Codes / Class rules
- Operational Experiences
- Technology developments

2016 Offshore Structural Reliability Conference

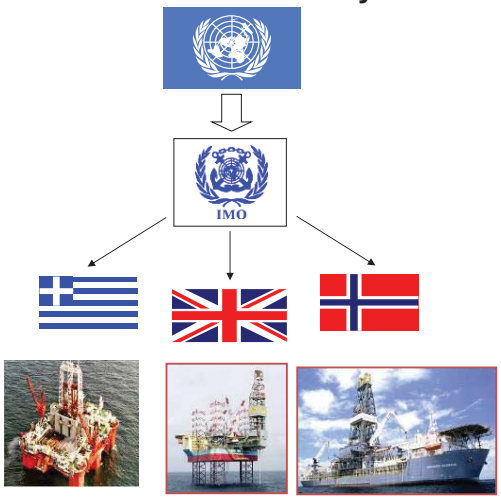
Regulatory basis for Mobile Offshore Units (MOUs)

- With few exceptions MOUs follow a maritime safety regime
- Major parties:
  - National Maritime Authorities (flag states)
  - Port State Authorities
  - Classification Societies
  - National Coastal State Authorities (shelf states)

Regulatory basis for Mobile Offshore Units (MOUs)

- Compliance with the maritime safety regime is documented by certificates:
  - Maritime certificates from a flag state
  - Classification Certificates from a recognised Classification Society
- Confirms Compliance with:
  - IMO Conventions and Flag state regulations
  - Classification rules/standards
- Additional requirements will normally apply from coastal states where the unit is operating

International Maritime Safety Regime; – Structure and hierarchy



- IMO is United Nation's body for maritime affairs
- Develops and maintains conventions giving safety regulations for ships (and MOUs) trading internationally
- Abt. 156 nations are members of IMO
- Secretariat is and meetings take place in London
- IMO has no operational role

- Maritime Safety Regime is a Flag State Regime as the Flag State is *enforcing internationally agreed safety regulations* (the conventions) through national maritime law

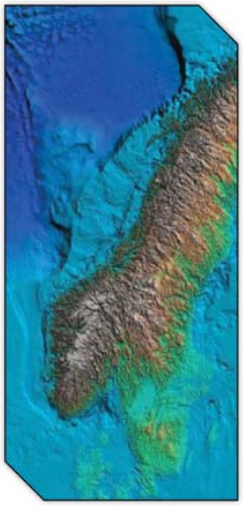
Relationship – Flag & Class

- An MOU operating internationally has to comply with the safety regulations issued by the Maritime Authority of the country whose *flag* the unit is flying (the Flag State)
- Flag states require classification
- Delegation of authority from Flag State to Class is normal

- For operation on continental shelves, additional local coastal state requirements apply

Coastal (shelf) State Legislation

- ***MOUs and FPSOs must comply with Coastal State Legislation of the country where it is to operate***
- All states have ***full sovereignty*** w.r.t regulating activities on their continental shelves
- Activities on the shelf are generally ***not*** regulated by any common ***international conventions***
- National legislation ***precede*** Maritime (Flag State) Rules
  - Will normally refer to flag state rules w.r.t. maritime aspects
  - Will occasionally require flag as well as class.
- Compliance procedures and systematics normally ***deviates*** from maritime practise



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Class Rules vs. International Legislation

International legislation covers aspects related to Safety, Health and Environment:

- Accommodation
- Life saving
- Navigation
- Fire
- Load Line, Stability
- Radio Communication
- Dangerous goods
- Security
- Pollution Prevention
- **Technical safety (Class certificate)**
- Manning …

Traditional Classification coverage

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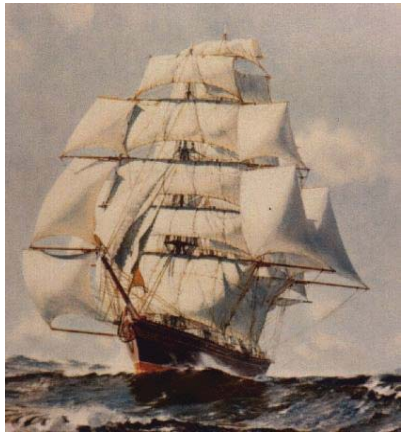
Classification Societies – Background

- Increase in international trade in the middle of the 18th Century
- Ship owners needed insurance to reduce their economical risk
- **Insurers** needed someone to:
  - establish safety standards for ships
  - verify that the standards were complied with
- Major Classification Societies established by marine insurers more than 150 years ago



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The Classification Concept

EXPERIENCE FEEDBACK

- Design Approval
- Construction survey
- Certification
- Operation follow-up

- Classification Societies :
  - set quality standards for hull, machinery and equipment
  - verify that ships and offshore units are built and maintained according to these standards
- Classification has gained world-wide recognition as representing an adequate level of safety and quality, by e.g:
  - maritime and coastal state authorities
  - insurance underwriters
  - owners
  - building yards and sub-contractors
  - finance institutions

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Development of Class Rules

Rules have evolved from being empirical, to being based on engineering principles and safety assessments

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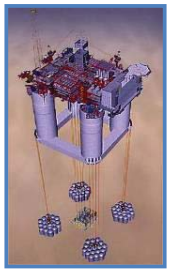
Types of Units covered by Classification Rules


Semi-submersible


Jack-up


Ship-shaped Unit


TLP (Tension Leg Platform)


Deep Draught Unit (DDU) (SPAR)


Loading Buoy

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Class Notations

- Scope of classification is determined by mandatory and additional *class notations*, e.g.

Construction symbol — ✠

Main character of class — **1A1**

Service and type notations — **Ship-shaped Oil Production and Storage Unit(N)**

System and equipment notations — **FMS, HMON, POSMOOR, ECO, CRANE, HELDK-SH, PROD(N)**

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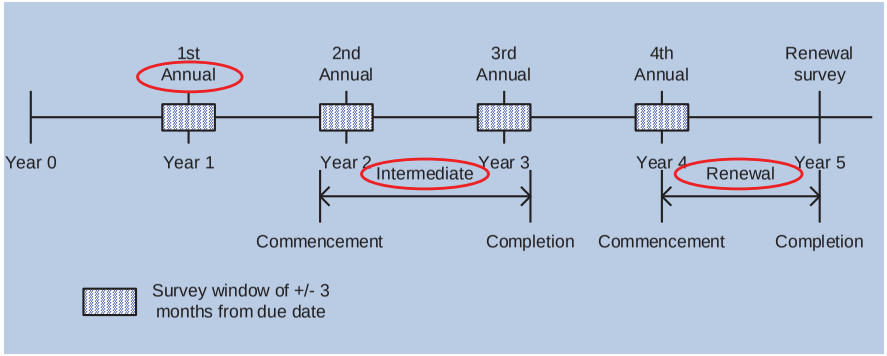
Class Surveys in Operation – Principles



Minimum frequency of inspection is set by IACS unified requirements

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Accidents have always influenced maritime rules and regulations

Titanic (1912)	Torrey Canyon (1967)	Herald of Free Enterprise (1987)	Exxon Valdes (1989)	Estonia (1994)	Erika (1999)
• SOLAS •First edition, 1929	• MARPOL (1973) • STWC (1978)	• SOLAS '90', • ISM	OPA 90 MARPOL	• SOLAS '95'	• MARPOL • Single hull oil tanker phase-out • EMSA

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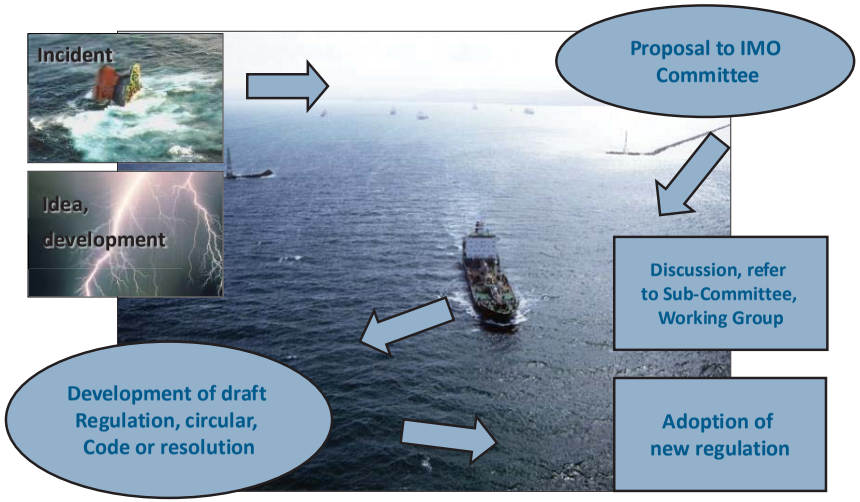
IMO Process - example



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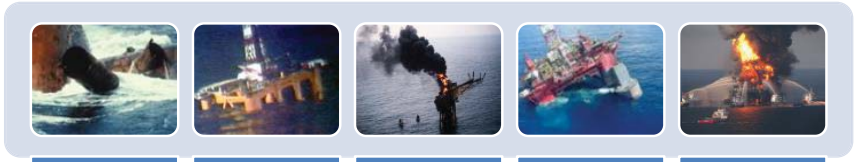
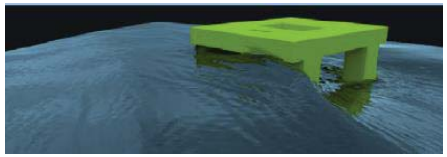
Offshore accidents that have changed rules/regulations



1980 Alexander Kielland, Norway	1982 Ocean Ranger Newfound-land	1988 Piper Alpha UK	2001 P36 Brazil	2010 Deepwater Horizon US GOM
Structural redundancy Survival floatability Leakage detection Rescue suits Lifeboat launching	Major rev. of IMO MODU Code - Ballasting - stability New Canadian safety regulations Evacuation equipment	Safety Case legislation ALARP Separation principles Firewalls Process safety	Risk assessments for non-standard operation Hazardous area classification Cascading of alarms	New regulator (BSEE) New Well control regulations Updated standards (API, NORSOK, Class) Update to IMO MODU Code

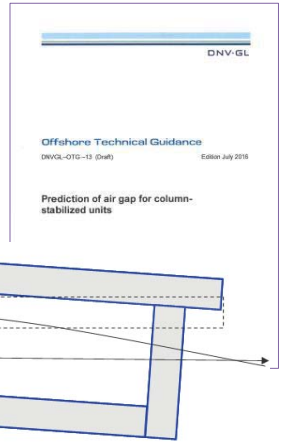
COSL Innovator Incident 2015

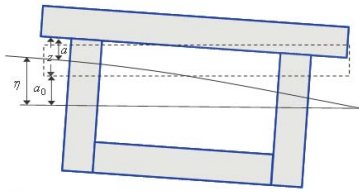
- COSL Innovator was was hit by a steep wave that caused extensive damage to windows in 17 cabins on two decks, with loss of one person's life.
- The COSL Innovator accident has provided new insight with regard to air gap and wave slamming
- New DNV GL guidelines are being prepared



Guideline for air gap predictions

- The objective of OTG-13 is to define a recommended procedure for estimating air gap for column-stabilized units.
- The procedure can be applied to predict air gap for operating conditions as well as design air gap with annual probability of exceedance q = 10^{-2} (ULS) and q = 10^{-4} (ALS).





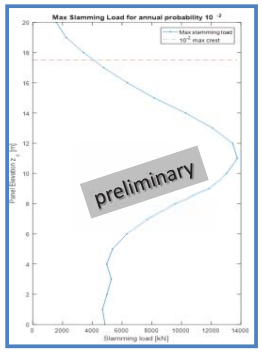
Air gap:

$$a(x,y,t) = [a_0(x,y) + z(x,y,t)] - \eta(x,y,t) = a_0(x,y) - \chi(x,y,t)$$

Still water air gap — Upwell

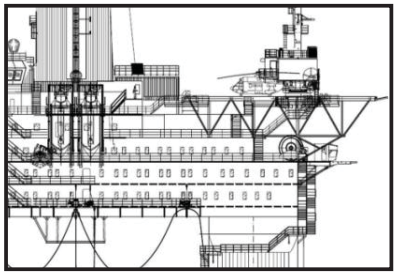
Guideline on horizontal wave impact loads (not issued)

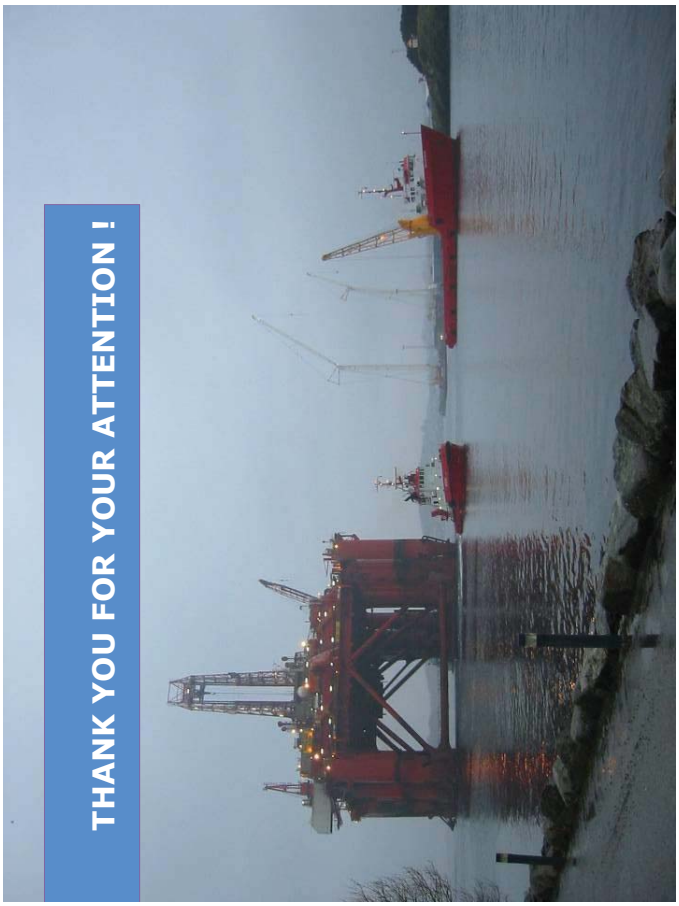
- The objective of OTG-14 is to provide a guideline for the loads to be used to document structural and floating integrity for MOUs which are subject to horizontal wave impact with the deck in the design conditions.

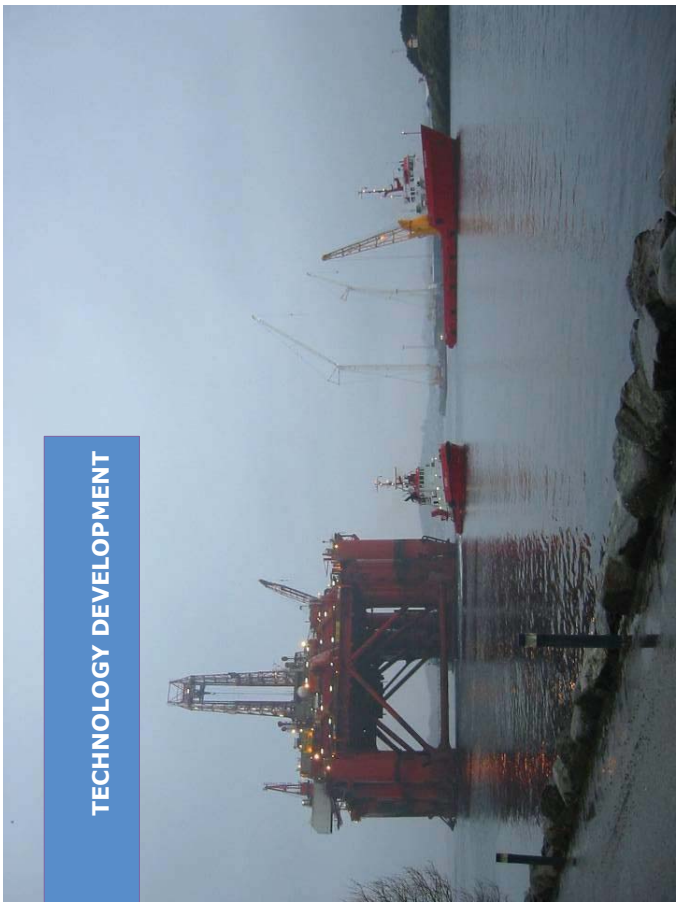


COSL Innovator Incident – Windows in cabins

- Following this PSA has issued letter confirming that cabins may be accepted without windows
- NMA have followed up by proposing changes to their Living Quarter regulations removing the requirement to natural light in cabins (currently on hearing)
- Also flag authorities state that they are positive to evaluating removal of windows in cabins on a case-by-case basis







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Extensive experience feed-back from Classification records

27 million functions and components

17 million components

428 thousand survey findings since 2005

50 000 findings per year such as damages, malfunctioning, non-conformities, overdue surveys

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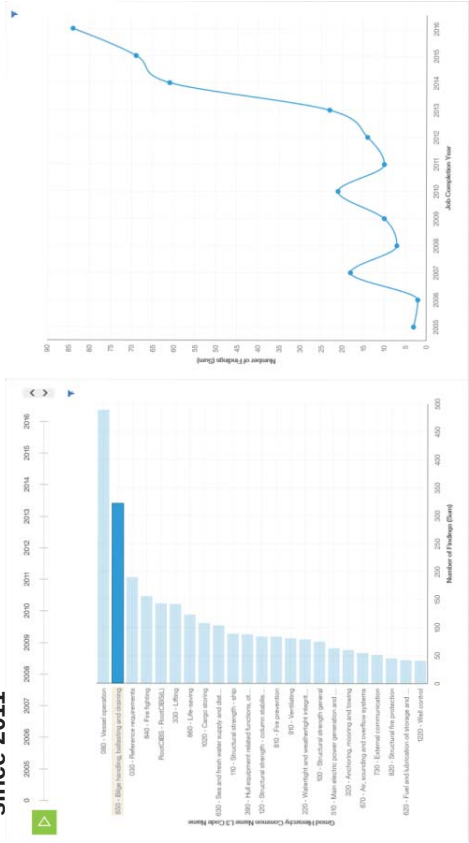
Example: Findings related to Bilge and Ballast shows significant increase since 2011



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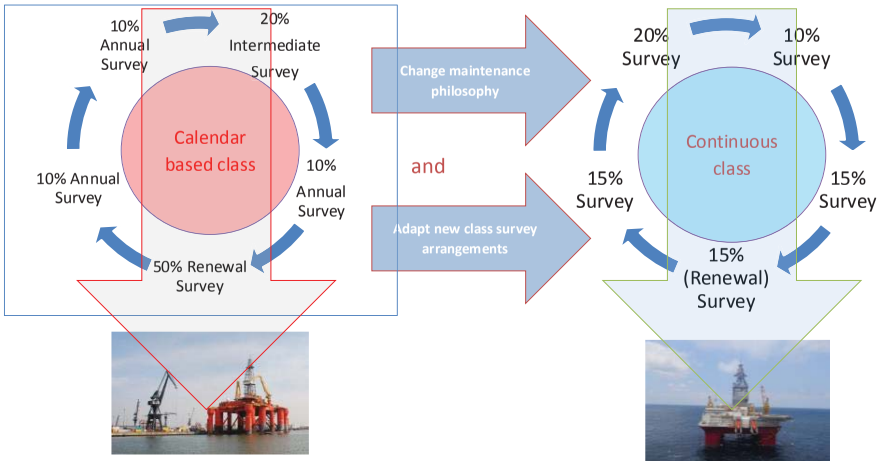
Continuous Class



Most class work left to 5 year Renewal

Continuous class done on location

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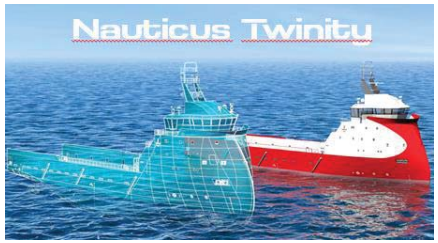
Utilising simulator technology for continuous class

Opportunities

- Establish a digital twin of the unit
- Move part of on-board activities to shore replacing the systems under control with a simulator
- All subsystems (i.e. PMS, DP, Drillfloor, ESD, F&G) may be simulated as a whole and integrated in Twinity as they would be on the vessel/installation
- Verify software upgrades
- Utilize DNV GL Surveyor with specialist competence
- Focus class activies onboard on specific areas



Minimize upset of operation

11.2 Presentation by Mike Hoyle

NTNU Norwegian University of Science and Technology

International Association of Oil & Gas Producers

Jack-Up reliability with reference to ISO 19905-1 & more

Mike Hoyle

Noble Denton marine services, DNV GL

The 3rd Offshore Structural Reliability Conference
OSRC2016
14-16 September, Stavanger, Norway

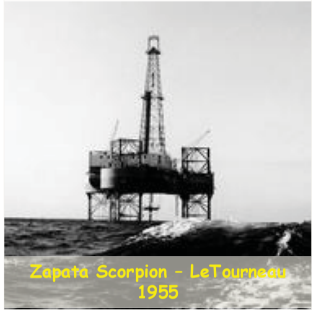
Content

- Introduction
- Site specific assessment
- Indicative reliability
- The risk background
- Unexpected survivals
- Emplacement & removal
- Conclusions
- References

William Arrol's Tay Bridge jack-up 1882

Introduction

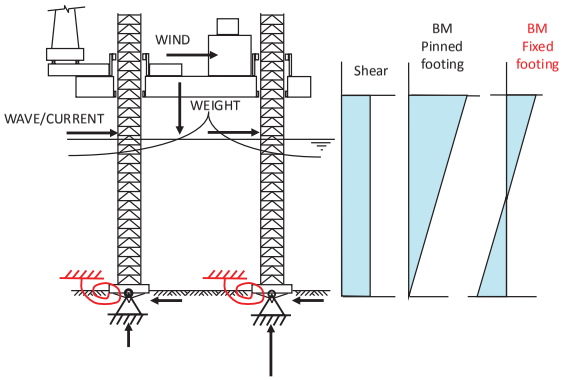
- Jack-ups move around - rarely designed for a specific location.
- Design usually to Class / IMO MODU code - sometimes with a passing thought to Site Specific Assessment requirements.
- Have operating manuals which include Classed limits, which may, or may not, be representative.
- Need to be evaluated for each location they visit, as a minimum with reference to the foundation.
- Are dynamic.



Zapata Scorpion - LeTourneau 1955

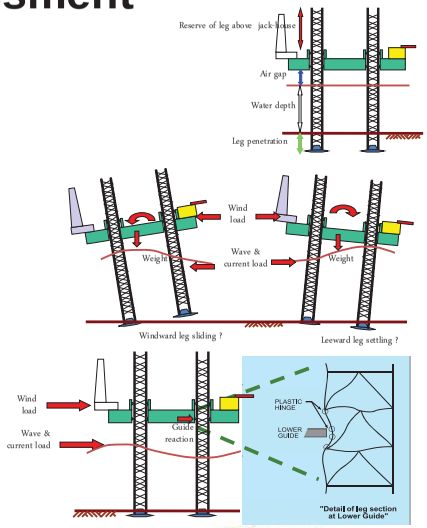
Introduction: Foundation fixity is the key

- Can halve the BM at the hull
- Increase the sway stiffness by a factor of 4 – and halve the natural period
- Dynamics are important and a function of foundation fixity
- But … foundation fixity can reduce as loads increase



Site specific assessment

- Assess:
  - Penetration/geometry/airgap (leg length)
  - & storm survival:
    - Preload, foundation (bearing & sliding),
    - Overturning,
    - Strength : Leg & leg-hull connection.
    - Rely on Class to ensure hull strength



Site specific assessment

- Normally no assessment of storm "ALS". Exceptions:
  - When de-manned in TRS areas – but return period (presently) to be agreed between the parties. (SNAME GoMex Annex & ISO; SNAME indicates 10-yr independent extremes)
  - Airgap to be sufficient to clear abnormal wave crest with j-p tide/surge. (ISO)

Site specific assessment

- Up until 1990's the approaches to SSA were somewhat diverse.
- Development of SNAME T&RB 5-5A,1st published 1994 [16], provided the first generally accepted approach [Jones et al 1993].
- SNAME updated over the years (now Rev 3 + 1 Corrigenda).
- ISO 19905-1 evolved from SNAME. 1st Ed 2012; 2nd Ed 2016.
- Both SNAME & ISO [11] use LRFD / PF approach.

$$\gamma_1 D + \gamma_2 L + \gamma_3(E + \gamma_4 D_n) \le \phi R \quad \text{[SNAME]}$$

$$\gamma_{f,G} G_F + \gamma_{f,V} G_V + \gamma_{f,E}(E_e + \gamma_{f,D} D_e) \le R / \gamma_R \quad \text{[ISO]}$$

$\gamma_1 = \gamma_2 = \gamma_4 = \gamma_{f,G} = \gamma_{f,V} = \gamma_{f,D} = 1.0$; range of L / G_V required.

Site specific assessment

- Historically jack-ups have been assessed using independent extremes at the 50yr return period level.
- SNAME uses LF, γ_3 = 1.15 (1.25 prior to 2002).
- ISO uses AF, $\gamma_{f,E}$ = 1.15 for 50-yr independent extremes.
  ISO uses AF, $\gamma_{f,E}$ = 1.25 for 100-yr joint-probability data.
- SNAME was calibrated against exemplary rigs at their limits when assessed to best practice, based on component level checks [Ahilan, Baker & 1993]
- Based on average reliability of all the rigs (not all their limits for all checks) LF =1.25. Based on lowest reliability rig LF = 1.15 as adopted in SNAME Rev 2, 2002.

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Site specific assessment

- A bunch of changes from SNAME to ISO [Hoyle et al, 2006]; the main differences in ISO are:
  - Abnormal airgap check
  - More/better/clearer guidance e.g. modelling & response [Tan et al, 2012]
  - Earthquake should be considered (simple(?) screening)
  - Apparent wave period effects to be included [Dowdy et a 2012]
  - Latitude dependent KRF [Hoyle et al 2009 & Stiff et al 2012]
  - Improved foundation stiffness and capacity modelling [Wong et al, 2012]
  - Less onerous column curve for HSS [Frieze, 2012].
- Benchmarking of ISO v/s SNAME indicated that ISO is generally slightly less conservative than SNAME [Stiff et al, 2012].

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Indicative reliability

- Pushovers show RSR >2.0 if foundation is stable (some 20 cases).
  [Morandi, Jin, Smith (2001) & HSE (2003)]
- SNAME (Rev 2) & "North Sea Annex"=ISO, load and resistance factors safeguard the foundation against 10,000 year overload (for CNS & SNS met data).
  [Meyer, Hoyle, Williams & Jones (2003)]
- An investigation into sliding failure indicated that when SNAME Rev 1 factors are met, sliding would initiate in 10,000 year INDEPENDENT extremes in CNS.
  [Hoyle & Snell (1997)]

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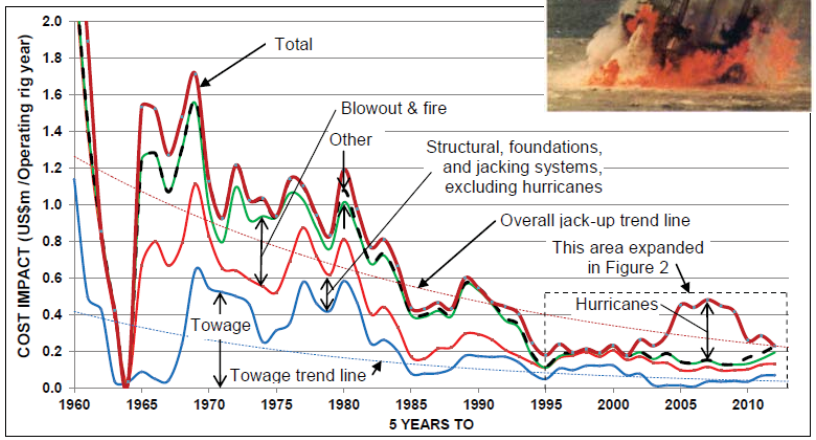
The risk background



Jack-up Accident Statistics – a Further Update, R.L Jack, M.J.R. Hoyle, N.P. Smith & R.J. Hunt, The Jack-Up Drilling Platform, City University, 2013

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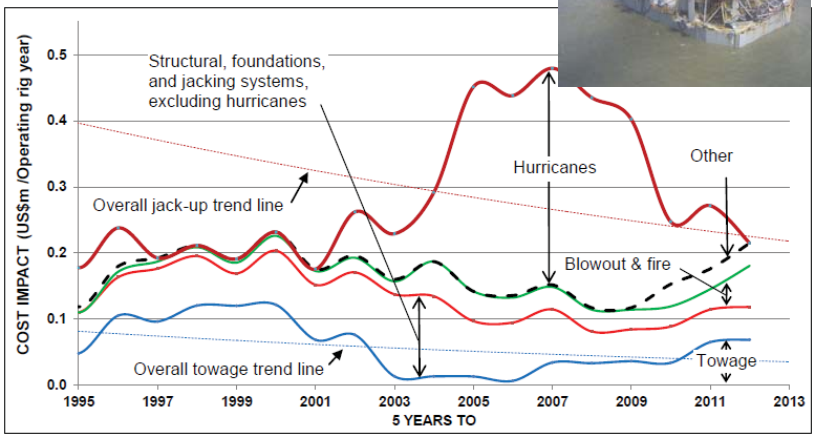
The risk background



Jack-up Accident Statistics – a Further Update, R.L Jack, M.J.R. Hoyle, N.P. Smith & R.J. Hunt, The Jack-Up Drilling Platform, City University, 2013

The risk background

- Jack-ups have suffered no unexpected storm-induced failures.
  Remember that the GoMex philosophy used to be: "keep the people safe and don't worry (too much) about the jack-up".
- Have seen unexpected survivals
  [Hoyle & Brekke, 2006 & Templeton, Lewis & Brekke, 2009].

➔ Our focus has been on in-place survival; the risk record shows that it should embrace other issues:
- Towage – this is not for ISO/TC67/SC7
- Emplacement & Removal from site
- Maintenance, blow-out & fire.

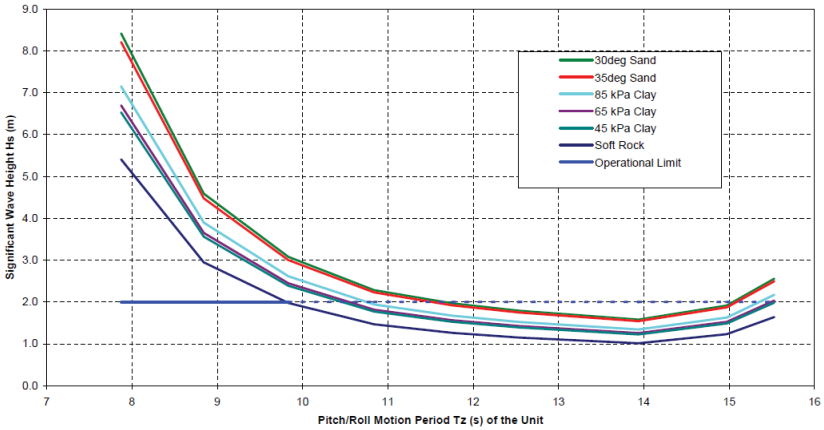
Unexpected Survivals

- Back analyses show that a number of jack-ups exposed to hurricanes survived when they would not have been expected to do so [Stock et al 2000; Hoyle & Brekke 2006; Templeton et al 2009].
- Given that some of the analyses were undertaken some years back, they will not have used the latest approaches.
- Nevertheless, aggressive adjustments to the assumptions were required in order to show the possibility of survival.
- Some of the units were instrumented and the T_n's indicated stiffer behaviour than expected.
- Based on the above it seems that our site assessments are conservative.
- We should re-visit these cases using today's technology.

Emplacement & removal

- No standardised approach or requirements.
- No formal documentation on who should supply what information, and when. Some Operators shirk their responsibility and provide poor information too late.
- IMO & some Class require that limits to e.g. seastate for emplacement be specified.
  Such limits often empirical and/or very conservative and/or confusing e.g.:
  "Wave height less that 1.5m and period less than 6 sec"
- Either: Followed – at the cost of delays and prolonged exposure,
  Or ignored and "driving blind" …
  Or we can do something better that accounts for foundation, leg length below hull, etc. …..

Emplacement & removal - Emplacement limits

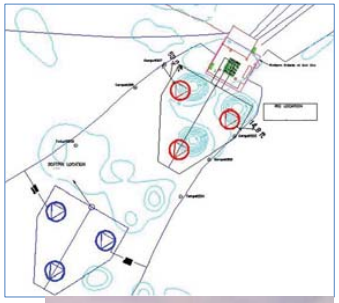


Emplacement & removal - Other considerations

- Practical Operability
  - Centre of Buoyancy compatible with transit LCG
  - Unless the transit CoG and the Hull CoB are aligned, time can be wasted adjusting the CoG when the hull is being elevated out-of or lowered into the water.
  - If not done carefully, there is potential for pinion overload or severe trim.

Emplacement & removal - Other considerations

- Non-storm loadcases to provide operational robustness
  - Resilience against leg sliding, settlement/punch-through, boat impact, leg extraction.
  - These events and “abuse” do happen.
  - Ensure that the leg (braces) can sustain reasonable resulting load levels and
  - The rig can still jack without causing damage.



Emplacement & removal - Solution – ISO 19905-4

Jack-up emplacement and removal at a site

- Will identify and address the challenges with the aim of improving safety and reducing the potential for:
  - damage to assets and
  - conflicts between the parties.
- Drafting in progress; many interested organisations & persons.
- Topics include:
  - Planning
  - Data requirements to/from rig owner & site operator
  - Communications
  - Roles & responsibilities
  - Competence
  - Potential causes of damage and their mitigations
  - Guidance on consistent determination of a rig’s limitations at a site.

Conclusions

- ISO 19905-1 provides reasonable structural and foundation reliability for jack-up elevated operations. But there is more to learn from past events - and from ongoing research.
- There are other risk-areas that should be addressed.
- ISO 19905-4 will address emplacement and removal from location, tow of the highest risk operations for a jack-up.
- Drilling, maintenance & towing risks are outside the ISO/TC67/SC7 scope – but should be addressed.

Concern: The current down-turn will bleed much-needed competence from the jack-up industry.

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Chapter 12

Session 10: Stability of Floating Platforms in a Reliability Perspective

12.1 Presentation & article by Xiaozhi (Christina) Wang, Amitava Guha & Qing Yu

NTNU Norwegian University of Science and Technology

International Association of Oil & Gas Producers

Assessment of Intact and Damage Stability Regulations for Offshore Floating Structures - in Reliability and Risk Perspectives

Xiaozhi (Christina) Wang
American Bureau of Shipping

The 3rd Offshore Structural Reliability Conference
OSRC2016
14-16 September, Stavanger, Norway

Introduction

- Stability Criteria in the IMO regulations are largely driven by prescriptive rules
- As an alternative to the quasi-static approach, the IMO MODU Code also adopted a performance-based intact stability criterion for Column Stabilized Units
- In addition to the stability during transit (free-floating), offshore structures with mooring lines and risers attached are also designed for in-place stability
- Dynamic stability is considered in certain types of offshore structures but is in general less of a concern than for ships
- Damage causes and scenarios for offshore structures differ in many ways from those of ships
- Reliability analysis requires a well-defined calculation methodology and failure criterion that reflects the failure mechanism – both need to be developed for stability assessment

Important Thought and Quote

- In 1957, Rechtin, Steele, and Scales of Bethlehem Steel presented their seminal SNAME Transactions paper:

Engineering Problems Related to the Design of Offshore Mobile Platforms

By E. C. Rechtin,[1] Member, J. E. Steele,[2] Associate Member, and R. E. Scales,[3] Visitor

- The paper describes how ship rules have evolved over the years to arrive at a relatively safe approach, but one that is largely empirical
- For wave loads, the paper goes on to say, engineers in the offshore industry have had to develop calculation techniques from “first principles”
- However, for stability, there was no theory that could be adapted, so the empirical ship rules were modified and applied to offshore structures
- This has generally been successful, as this paper will show
- However, it is not easy to quantify the reliability, as will also be shown

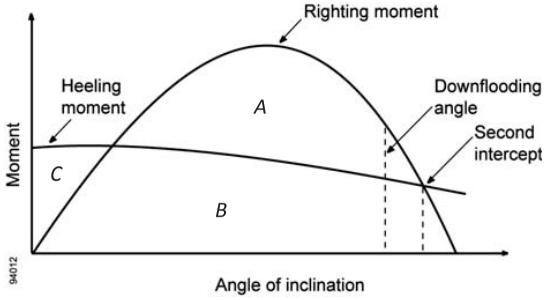
ABS MODU Rules and FPI Rules

- The intact stability criteria are based on the assumption of steady wind and calm water
- In 1986, ABS led a JIP to develop the dynamic-response-based criteria in reaction to the loss of two semi-submersibles
- The IMO MODU Code adopted the ABS dynamic-response-based criteria as an alternative intact stability criterion in 1989 Res. A649 (16)
- The current IMO requirements remain the same as in 1989. Ref. IMO MODU Code Res. A.1023(26) (2009)
- Damage stability criteria are required to avoid further flooding and maintain residual stability after each assumed damage or remote flooding occurs

Intact Stability Criteria

- Area Ratio Criterion

$\frac{\text{Area}(A+B)}{\text{Area}(B+C)} \geq 1.3$ or 1.4 depending on the type of the hull



Ref. IMO MODU CODE 2009, Figure 3-1

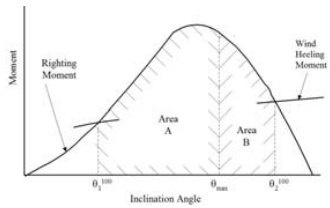
Interpretation In a Reliability Perspective

- Area Ratio Intact Stability Criterion
  - The limit state function comprises the quasi-static components
  - Statistics of restoring moment and overturning moment can be determined and are dependent on the hull configuration
  - **However**, reliability calculations and results are only meaningful when there is a well-defined calculation methodology that has well-defined failure criteria
  - The calculation methodology in the Area Ratio approach is well-defined, but there is no "realistic" failure mechanism, only a failure to meet the criteria
  - Applying reliability techniques to the classic "Area Ratio" approach therefore will give an excellent answer to the question "What is the probability that I fail the area ratio test?", but that has no significance to the probability of actually capsizing

Intact Stability Criteria

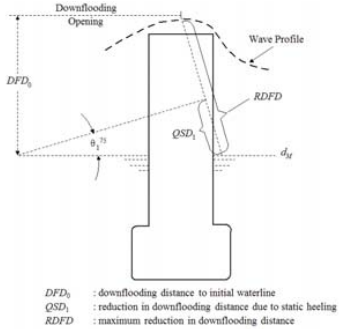
- Dynamic-Response-Based Criteria
  - Capsize Criterion

    $\frac{\text{Area}(B)}{\text{Area}(A)} \geq 0.1$
  - $\theta_{\max}$: maximum dynamic heel angle about the axis of rotation



Ref. ABS MODU Rule, 3-3-A1/Figure 1

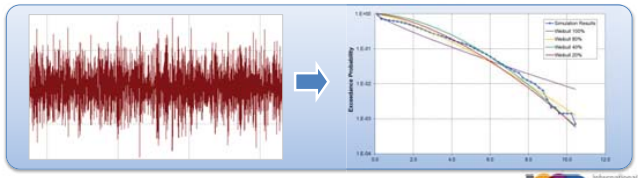
-- Downflooding Criterion



Ref. ABS MODU Rule, 3-3-A1/Figure 2

Interpretation In a Reliability Perspective

- Dynamic-Response-Based Intact Stability Criteria
  - Statistics of quasi-static restoring moment and overturning moment are dependent on the hull configuration
  - The dynamic response, i.e. the maximum heel angle, is determined through the motion analysis of the floating structure subject to stochastic environmental conditions
  - The statistics of the maximum heel angle can be derived based on numerical analysis
  - Again, these items can easily be assessed using formal reliability techniques, but the meaning of the results are much less clear

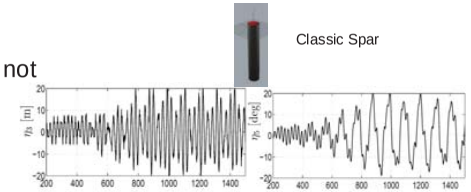


Proposed IMO 2nd Generation Intact Stability Assessment Criteria

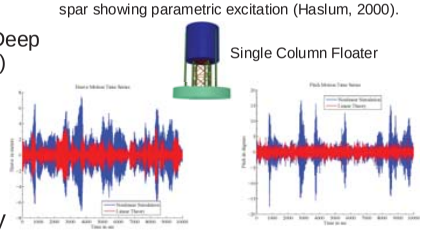
- The second generation criteria deals with five failure modes:
  - Broaching: Unable to keep course, often caused due to surf riding
  - Pure Loss of Stability: Prolonged reduction of static restoring forces when crest is close to amidships
  - Parametric Roll: Variation in roll restoring moment in longitudinal waves
  - Dead Ship: When ship looses its propulsive power and suffer beam wind and wave
  - Excessive Acceleration: Ship with very high GM has small natural roll period causing excessive acceleration
- Each failure modes will be assessed in three levels
  - 1st Level: Prescriptive expressions with high safety factor
  - 2nd Level: Analytical approximations
  - 3rd Level: Direct stability assessment using numerical methods

Dynamic Stability of Offshore Structures

- Dynamic stability is traditionally not considered in the design of offshore structures
- Parametric excitation has been observed in certain types of offshore structures and may cause significant motions
  - Coupled heave-roll/pitch motion of Deep Draft Vessels (spar, deep-draft semi)
  - Coupled surge-sway-yaw motion of moored FPSOs
  - Parametric roll motion of large multi-column semi in transit draft
- Probabilistic methods developed for the ship parametric roll analysis may be applicable



Model test result of heave and pitch motion of classical spar showing parametric excitation (Haslum, 2000).



Nonlinear simulation of SCF showing parametric excitation (Guha et al. ,2016)

Effect of Mooring on Stability

- Current requirements
  - Not to include the effect of mooring in the stability analysis
  - In some regulations and class Rules, the mooring effect unfavorable to stability must be taken into account
- There are industry initiatives to develop new stability criteria for 'permanently' moored floating production installations
- Numerical tools are available to include the dynamics of mooring in analyzing motions and freeboard and their statics in irregular sea
- Experience from developing the dynamic-response-based intact stability criteria can provide valuable input
- Technical challenges
  - Acceptance criterion and its application boundary
  - Site-dependent mooring capacity variation during the service life
  - Increased risk level of mooring failure that could lead to not only losing stationkeeping but stability (including the possibility that the vessel drifts over the intact mooring lines which increase the overturning moment by applying a load at the level of the keel)
  - Further validation of analysis tools

In Addition to Reliability Interpretation…

- Damage Stability
  - Occurrence probability of relevant damage scenarios
  - Probability of losing stability for each damage scenario
    - Residual stability that can be assessed to account for reduction in stability, change in heeling moments, weight shift, angle of heel and progressive flooding
    - Rapid loss of stability caused by a large scale flooding due to extreme wave actions, catastrophic incidents, or bifurcation after reaching critical heeling angle
- Probabilistic risk-based approach becomes more relevant for damage stability assessment. The probability and extent of damage can be assessed through Quantified Risk Assessment (QRA)
- However, the assessment will still be based on the current methods for assessing the residual stability of the vessel, and its likelihood of capsizing given the assumed damage
- In this case, part of the reliability can be assessed through QRA, but the actual failure criteria are still not well-defined

Probabilistic Damage Stability Requirement

- IMO Res. A265 (1973) introduced the probabilistic damage stability method for passenger ships
- Resolution MSC.194(80) (2005) adopted the harmonized probabilistic damage stability regulation for both dry cargo ships and passenger ships
- Similar concept has been used in the offshore industry as part of risk assessment for structures, equipment and system
- However, stability is typically not considered in the risk assessment for offshore structures… Or should it be?

Let History Tell the Stories…Jackup

- Many jackups have been lost under tow - two classic sinking were the Dan Prince in Gulf of Alaska in December 1980 and Rowan Gorilla 1 in North Atlantic in December 1988
- Observations:
  - Jackups almost always have low freeboard and a very high GM
  - Most are vaguely triangular, so their motions are much more complex than those of a ship
  - In severe weather there tends to be a considerable amount of green water on deck
  - Most jackups that have foundered have simply gained too much weight through progressive flooding. They slowly increase in draft, get ever more quantities of green water on deck, and finally capsize or sink
  - However, jackups do not capsize in the intact condition due to a failure of intact stability

Let History Tell the Stories…Semi

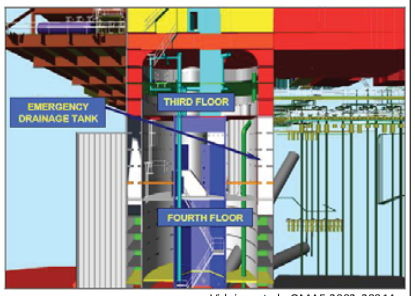
- On February 15, 1982, the Ocean Ranger semisubmersible capsized and sank on the Grand Banks near St. John', Newfoundland
- Observations:
  - A large wave caused porthole damage allowing seawater into the ballast control panel causing short circuit of the system
  - Erratic ballasting caused the platform to list and eventually capsize
- Should self preservation as a design objective for offshore platforms be considered?
- Reliability analysis of equipment in platform damaged condition needs to be performed?

Ocean Ranger's vulnerability to a rogue wave illustrated.
1 – For comparison, the Draupner wave 59 ft/18 m
2 – 28 ft/8.5 m
3 – Location of the ballast control room

https://en.wikipedia.org/wiki/Ocean_Ranger

Let History Tell the Stories…Semi

- On March 15, 2001, P-36 semisubmersible at Campos Basin had an accident that eventually sank the platform
- Observations:
  - Progressive flooding due to a sequence of events including: overpressure-induced rupture of emergency drainage tank; damage to equipment and piping; explosion of water-oil-gas mixture in the column; firefighting attempt and large listing causing further flooding
  - Damage scenario identification is critical to the application of the Rule-based damage stability criteria



Videiro, et al., OMAE 2002-28014

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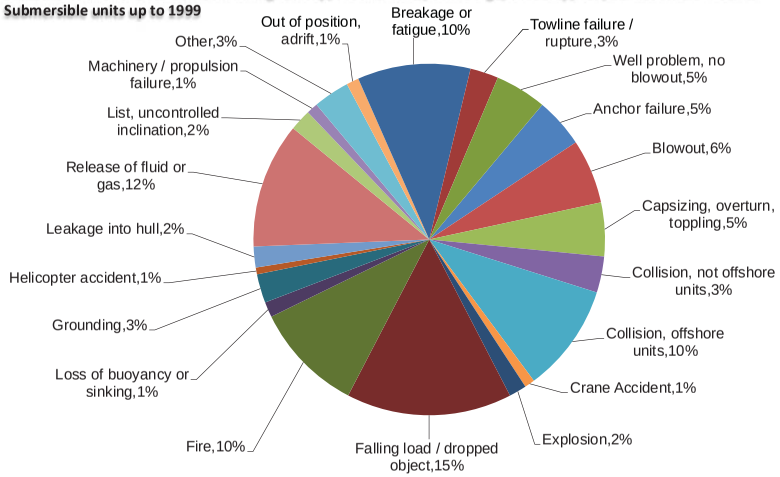
Causes of Platform Accidents

Statistics of offshore accidents involving Jack-up, Mobile Units, Drill Barge, Drill Ship, TLP, Semisubmersible and Submersible units up to 1999



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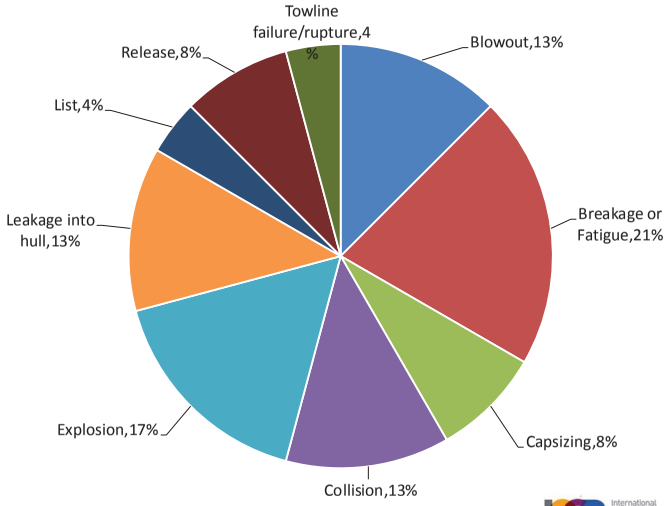
Top Offshore Accidents with Major Fatalities (1970-2007)



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Concluding Remarks

- Most of the "stability/sinking" failures that have occurred in the offshore oil and gas industry have not been due to a failure of the existing criteria, but due to other risks that are not part of the current stability calculation methodology. In almost all cases multiple failures were necessary for the results to be catastrophic
- Stability criteria for offshore structures are traditionally based on deterministic, quasi-static approach
- Dynamic-response-based approach has been developed but is adopted at difference paces by class societies, IMO, and local regulatory bodies
- Dynamic instability is believed less an issue for conventional offshore structure than for ships, but for certain types of platforms parametric excitation could occur similarly to what occurs for ships
- Reliability methods do not seem suitable for the stability analysis based on quasi-static responses, although the dynamic-response-based stability analysis provides a better way to quantify uncertainties
- Risk-based approach is gaining its way to the damage stability assessment for certain types of ships. For offshore structures, more industry awareness needs to be promoted

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Recommendations for Future Study

- Risk Assessments should take a far greater interest in potential events that could cause a stability/sinking impairment – The risk analyses are normally carried out anyway, so better advantage should be taken of the process to address stability issues
- Circumstances that will lead to actual stability failures need to be identified so that an actual failure criterion can be established
- Given a better understanding of the circumstances that will potentially lead to capsizing, then theoretically based calculations can be developed which can, in turn be subject to reliability assessment in order to determine the probability of failure
- Combine the probability of damage calculations, with the probability of intact "capsizing" and the probability of damaged capsizing (given the calculated probabilities of damage) and one can determine an overall probability of capsize failure
- In summary, stability calculations and criteria for offshore structures may be revisited with better understanding of the ACTUAL failure causes/mechanisms and the related consequences, and how to calculate for them
- Reliability based assessment of stability may become practical when response based criteria is explicitly defined

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Assessment of Intact and Damage Stability Regulations for Offshore Floating Structures - in Reliability and Risk Perspectives

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ABSTRACT: One of the main design objectives of floating offshore structures is the provision of a reasonable level of stability to enable safe operations. Several offshore structure failures, while intact or damaged, have occurred over the years in spite of this goal. This paper presents an overview of the cause of major offshore accidents and provides an insight into existing and potential future stability criteria. The challenges involving reliability and risk analysis in stability assessment are discussed, where a better understanding of the probabilistic failure mechanism is discussed, in addition to the deterministic based criteria.

KEY WORDS: Intact stability; Damage Stability; Response Based Criteria; Risk & Reliability

1 INTRODUCTION

Stability Criteria in the IMO regulations is largely driven by prescriptive rules. As an alternative to the quasi-static approach, the IMO MODU Code allows a performance-based approach for intact stability of column stabilized units [1]. In addition to stability during transit (free-floating), offshore floating structures with mooring lines and risers attached are also designed for in-place stability [2]. While dynamic stability is observed in certain types of offshore floating structures, it is in general less of a concern than for ships [3]. Reliability analysis requires a well-defined calculation methodology and a failure criterion that reflects the failure mechanism – both need to be developed for the reliability-based stability design.

The development of the design method for offshore floating structures during the nascent days of the offshore oil and gas industry is mostly based on the experience of designing ships. Ship rules have evolved over the years to arrive at a relatively safe approach, but one that is largely empirical [4]. For wave loads, engineers in the offshore industry have had to develop calculation techniques from "first principles". But for stability, the empirical ship rules were modified and applied to offshore floating structures. This has generally been successful, however, it is not straightforward to quantify the reliability as will be shown here.

The intact stability criteria in the classification rules issued by the American Bureau of Shipping (ABS) are based on the assumption of steady wind and calm water [5], [6]. In reaction to the tragic loss of two semi-submersibles, ABS led a JIP in 1986 to develop the dynamic-response-based intact stability criteria [7]. The IMO MODU Code adopted the dynamic-response-based criteria as an alternative intact stability criterion in 1989 [8] and continued this requirement in the IMO MODU Code 2009 [1].

Damage causes and scenarios for offshore floating structures differ in many ways from those for ships. Damage stability criteria mandate the prevention of further flooding and maintenance of residual stability after each damage or flooding event. It is important to learn from past offshore structure failures to develop a more reliable and robust stability criteria. When addressing key stability issues, it may be suggested that the "methodology" applied for the reliability based performance assessment for damaged ships such as Lee et al. (2006) [9] could be applicable for analysis of damaged offshore structures as well.

2 HISTORY OF OFFSHORE STRUCTURE FAILURES

A review of the major accidents involving offshore structures was performed to gain an insight into the common causes of failures. The various causes of offshore casualties involving jack-up, mobile unit, drill barge, drill ship, tension leg platform, semisubmersible and submersible unit recorded up to 1999 is shown in Figure 1. Focusing on the loss of stability while intact or damaged, the combined percentage of failure is over 10% which can be further broken down to capsizing, overturn and toppling, 5%; loss of buoyancy or sinking, 1% ; leakage into hull, 2% ; and list and uncontrolled inclination, 2% .

The report by OGP [10] on major accidents involving loss of life provides the initiating causes of failure as shown in Figure 2. Relevant details pertaining to the causes of some of the major accidents are summarized in the following subsections.

2.1 Drill Ships and Barges

The *Glomar Java Sea* capsized while experiencing 75 knot winds over the bow and a list of 15° starboard. The vessel sank within minutes in 317 ft of water in October, 1983. In May, 1985, the *Tonkawa* drill barge sank due to a human error wherein the crew failed to close the ballast inlet valve and the ballast control operator was incorrectly supervised [11]. The non-drilling barge *Concem* capsized in calm weather in November 1985 due to a loss of stability near Stavanger, Norway [12]. The *Viking Explorer* capsized due

to an explosion in South East Borneo in September, 1988. The lift boat *Avco V* encountered heavy seas in the Gulf of Mexico in July 1989 and capsized due to an inadequate stability margin [13]. The drill ship *Seacrest* capsized in heavy seas during Typhoon Gay with 100 mph wind in the Gulf of Thailand in November 1989.

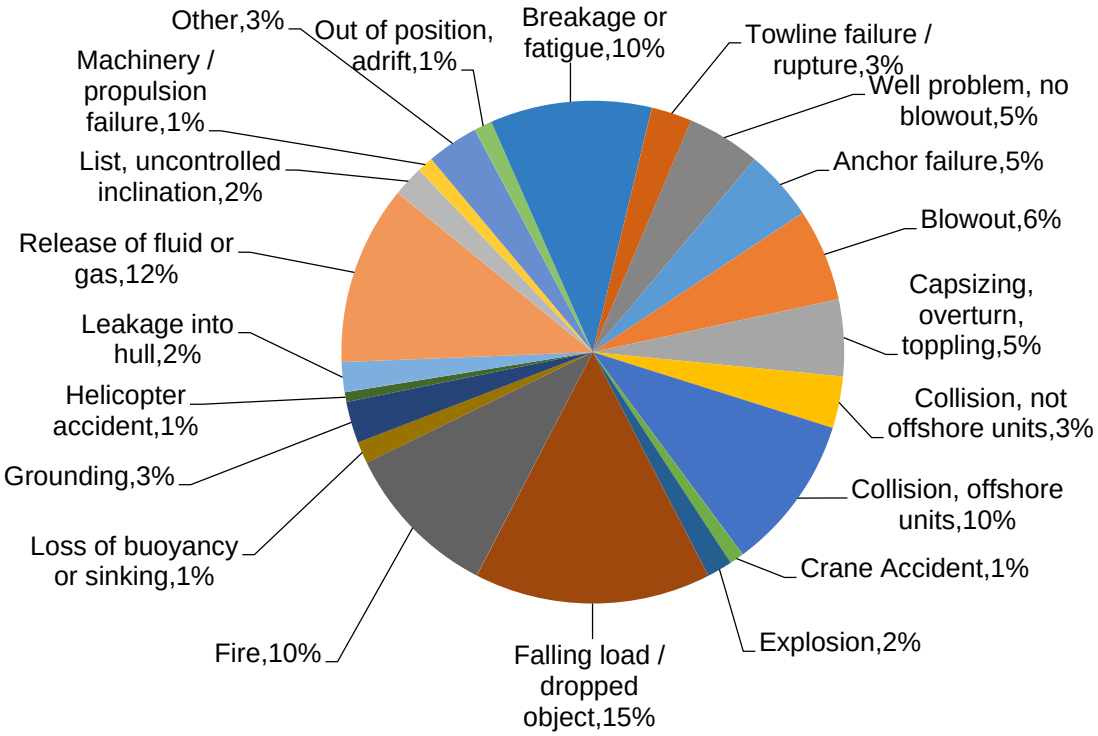


Figure 1. Statistics of offshore accidents involving Jack-up, Mobile Units, Drill Barge, Drill Ship, TLP, Semisubmersible and Submersible unit up to 1999

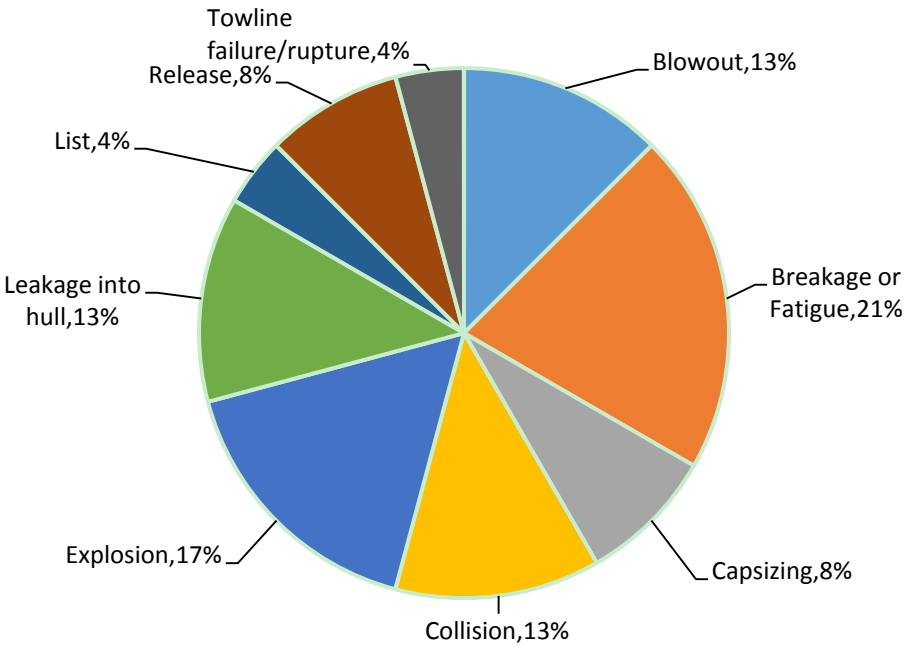


Figure 2. Major offshore incidents involving loss of life (1970-2007), OGP(2010) [10]

2.2 Jackup rigs

Many jackups have been lost while in tow. Two classic sinkings were the *Dan Prince* in the Gulf of Alaska in December 1980 [14] and *Rowan Gorilla 1* in the North Atlantic in December 1988 [15]. Jackups almost always have low freeboard and a very high GM. Most are vaguely triangular, so their motions are much more complex than those of a ship. In severe weather there tends to be a considerable amount of green water on deck. Most jackups that have foundered have simply gained too much weight through progressive flooding. They slowly increase in draft, get ever more quantities of green water on deck, and finally capsize or sink. Failure due to loss of stability in intact condition, however, has not been reported so far.

2.3 Semisubmersibles

On February 15, 1982, the Ocean Ranger semisubmersible capsized and sank on the Grand Banks near St. John', Newfoundland. A large wave inflicted porthole damage and allowed seawater to come into contact with the ballast control panel, causing a short circuit of the system. Erratic ballasting caused the platform to list and eventually capsize.

On March 15, 2001, the P-36 semisubmersible at Campos Basin had an accident that sank the platform [16]. Progressive flooding occurred due to a sequence of incidents including an overpressure-induced rupture of emergency drainage tank; damage to equipment and piping; explosion of the water-oil-gas mixture in the column; a failed firefighting attempt; large listing causing further flooding and eventual sinking of the platform.

From the above case studies and statistical data, it is seen that most accidents are caused by human error and a departure from operational guidelines. A similar conclusion is made in the review of offshore structure failures in Ismail (2014) [17]. In addition to measures to reduce human errors and their impact, redundancy requirements in the design of offshore structures may also require further improvements. Damage scenario identification is critical to the application of the rule-based damage stability criteria. The structural stability and equipment performance in damaged condition needs to be assessed with a reliability perspective to better understand their potential consequences and foresee various damage scenarios.

3 THE CURRENT STATUS OF STABILITY ANALYSIS METHODOLOGY FOR OFFSHORE STRUCTURES

The intact stability criteria for offshore structures are developed in the same manner as that of ships. Due to the significant difference in the primary shape of the structure compared to ships, and hence their corresponding motion behavior at sea, the stability assessment using the righting lever curve may not lead to optimal designs.

3.1 Intact Stability Criteria

The common intact stability criteria are based on a quasi-static analysis method, known as the Area Ratio Method [6]. Figure 3 shows a typical righting moment curve and heeling moment curve, where the area enclosed within is used to define the stability criteria. The alternative approach is to demonstrate stability through the Dynamic-Response-Based criteria (Figure 4), where the maximum healing angle θ_{max} can be obtained through numerical simulations or empirical equations for a specific type of offshore floating structure. It must be noted that these stability criteria already include a safety factor and failure to meet the criteria should not be confused with physical failure of the platform stability.

3.2 Stability in Perspective of Reliability Analysis

For the reliability analysis, the limit state function based on the simple Area Ratio Method as described in Section 3.1 comprises the quasi-static components. Statistics of restoring moment and overturning moment can be determined and are dependent on the hull configuration. However, reliability calculations and results are only meaningful when there is a well-defined calculation methodology that has well-defined failure criteria. The reliability calculation in the quasi-static intact stability approach based on the Area Ratio Method is straightforward, but the inequality used in the assessment does not define a realistic failure mechanism but merely a failure to meet the criteria. Applying reliability techniques to the classic Area Ratio Method therefore will answer the question "What is the probability that I fail the area ratio test?", but that has no significance to the probability of actually capsizing.

In the Dynamic-Response-Based intact stability criteria, the statistics of quasi-static restoring moment and overturning moment are dependent on the hull configuration. The dynamic response, i.e. the maximum heel angle θ_{max}, can be determined through the motion analysis of the floating structure subject to stochastic environmental conditions in accordance with the ABS MODU Rules [6]. For example, Figure 5(a) shows a time history of the roll motion of a platform. We can obtain the probability of exceedance plot by fitting the time history data to a suitable distribution as shown in Figure 5(b) and obtain the maximum heel angle θ_{max}. Although these items can easily be assessed using formal reliability techniques, the meaning of the results are yet to be justified.

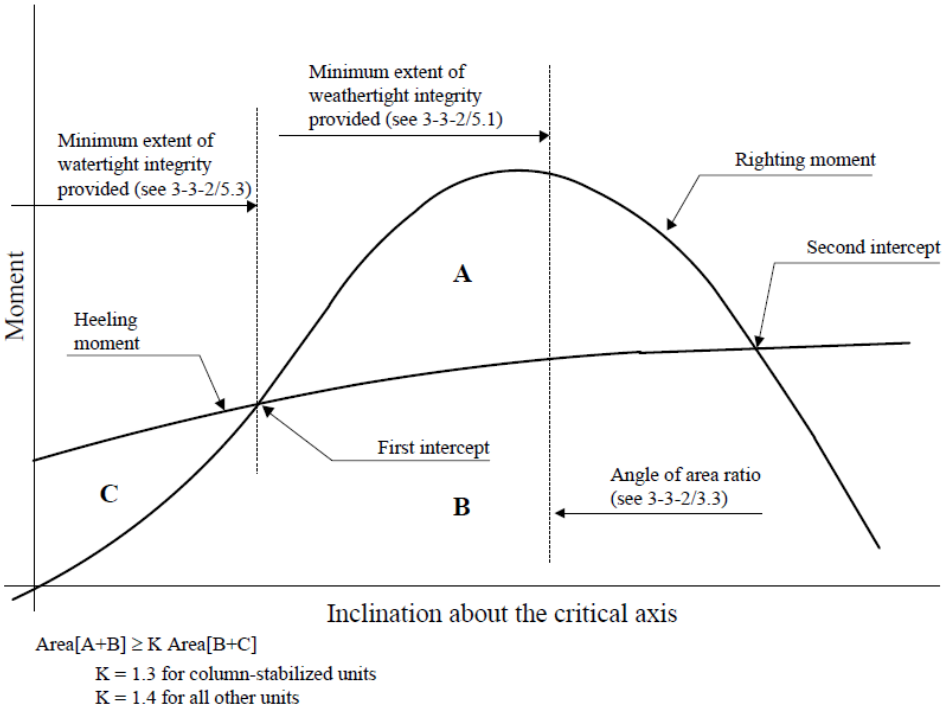


Figure 3. ABS Intact stability curve (ABS MODU Rule [6], 3-3-2/Figure 1)

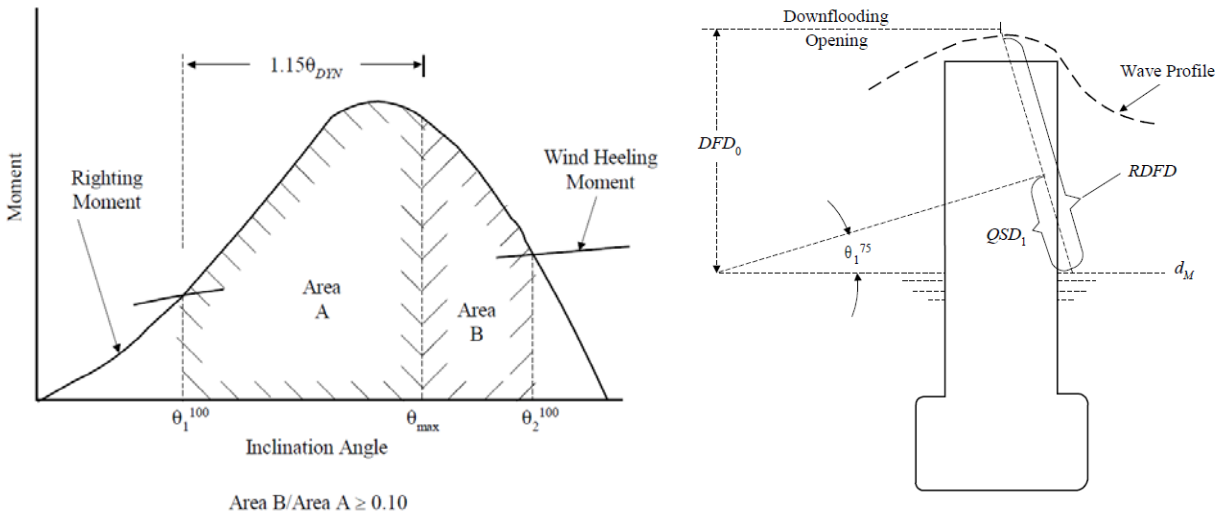
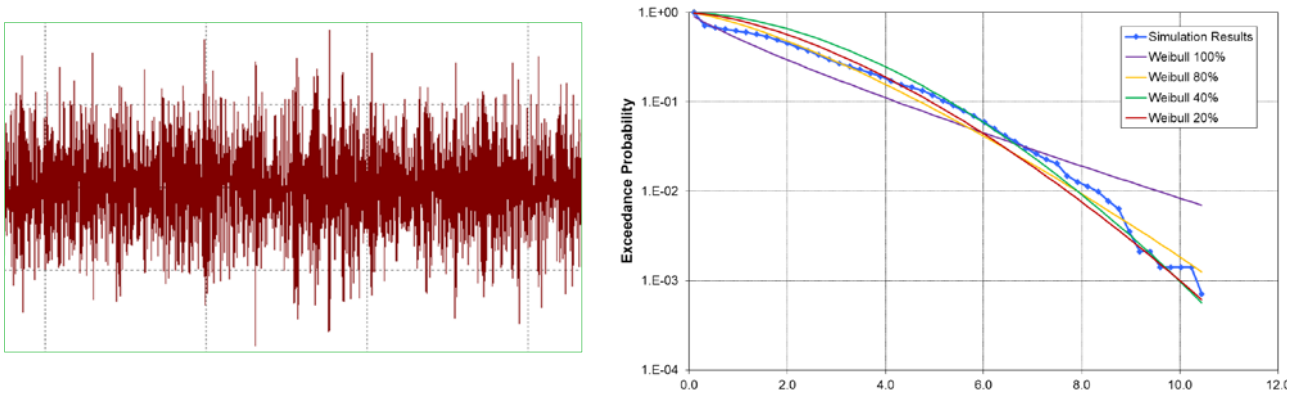


Figure 4. Capsize criteria based on dynamic response (ABS MODU Rule [6], 3-3-A1/Figure 1, 2)



(a) (b)

Figure 5. Probability of exceedance of heel angle obtained using numerical time history data

3.3 Effect of Mooring on Stability

The IMO MODU Code [1] does not consider the effect of mooring restoring moments in the stability analysis. In some regulations and class Rules, any mooring effect unfavorable to stability is required to be taken into account. There are industry initiatives to develop new stability criteria for permanently moored floating production installations. Numerical tools are available to include the dynamics of mooring in analyzing motions and freeboard and their statistics in irregular seas. Experience from developing the dynamic-response-based intact stability criteria can provide valuable input. The key challenges are: (1) Acceptance criteria and its application boundary; (2) Site-dependent mooring capacity variation during the service life; (3) Increased risk of mooring failure that could lead to not only stationkeeping failure but loss of stability, including the possibility that the vessel drifts over the intact mooring lines which increase the overturning moment by applying a load at the level of the keel; (4) Further validation of analysis tools required to establish confidence in the global performance results.

The current limitations of the intact stability criteria, some of which are discussed above, are the key motivation for the development of the second generation IMO code for intact stability for ships. The new criteria will be more physics-based than empirical, hence applicable for a wider range of structures. Under this development, five failure modes are primarily being investigated [18]:

i) Pure loss of stability

ii) Parametric rolling

iii) Broaching

iv) Harmonic resonance under deadship condition

v) Excessive acceleration

Although the current focus is on determining suitable intact stability criteria considering the above dynamic stability related problems for ships [19]–[22], once developed they will affect the direction of the future development of intact stability for MODUs as well [23]. Among the above failure conditions, the parametric excitation may also need to be considered for offshore structures.

4 PARAMETRIC EXCITATION IN DETERMINING STABILITY

Parametric excitation is traditionally not considered while evaluating intact stability in the design of offshore floating structures. It is however an area of recent interest to incorporate such analysis in the second generation of IMO regulation [18]. Currently, classification guidance exists for addressing parametric excitation of ships [24]. In the case of offshore structures, the following cases have been observed that could cause concerns:

- Coupled heave-roll/pitch motion of Deep Draft Vessels (spar, deep-draft semi) [25]
- Coupled surge-sway-yaw motion of moored FPSOs [26]
- Parametric roll motion of large multi-column semi in transit draft [27]

The parametric excitations problems are generally modeled as Mathieu equations and solutions presented as Ince-Strutt diagrams with stable and unstable regions as shown in Figure 6 [28]. The previous model tests by Haslum have shown susceptibility of classic spar platform towards such parametric excitation (Figure 7) [25], [29]. Another example of the adverse effect of parametric excitation in offshore structures is the Single Column Floater (SCF). The platform is susceptible to parametric excitation as shown in Figure 8 , where the linear vs.

nonlinear motion prediction is compared [30]. For the SCF, it was found that Mathieu type instability was a major concern and, as a result, the project was abandoned [31].

The probabilistic method developed for the ship parametric roll analysis is believed to be applicable for analysis of offshore structures as well. This however needs to be further studied and tested before industry application.

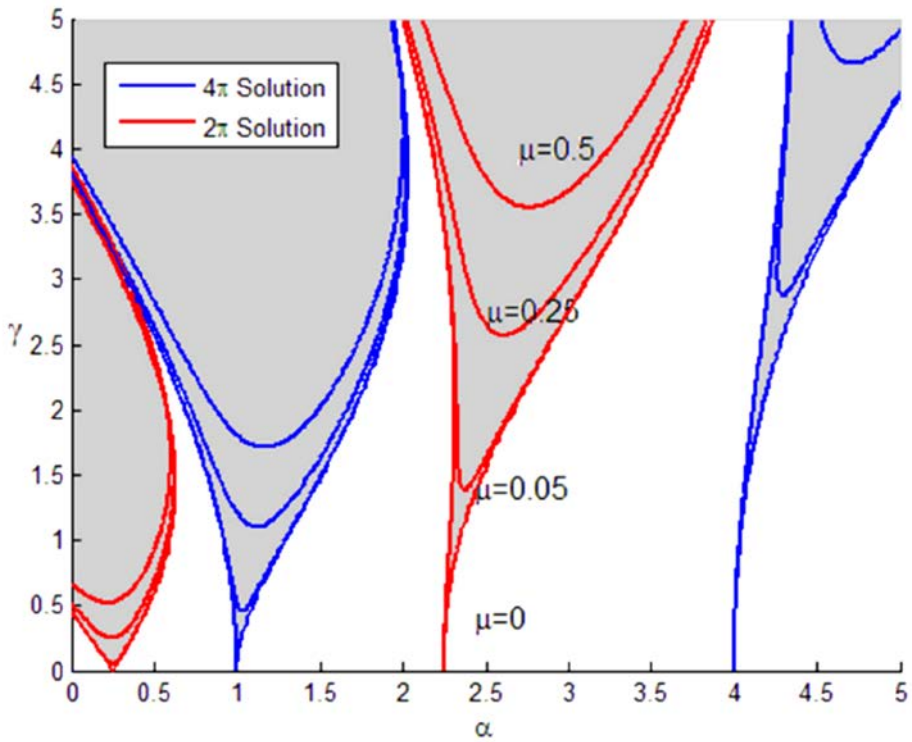


Figure 6. Ince-Strutt diagram showing stable and unstable solutions to the Mathieu equation[28]

5 PROBABILISTIC DAMAGE STABILITY ANALYSIS

The probabilistic risk-based approach has become more relevant for damage stability assessment. The probability and extent of damage can be assessed through Quantified Risk Assessment (QRA). The quantified evaluation of risk is generally defined as:

i. Occurrence probability of relevant damage scenarios
ii. Probability of losing stability for each damage scenario

Residual stability can be assessed to account for the reduction in stability, change in heeling moments, weight shift, angle of heel and progressive flooding. Also, the rapid loss of stability caused by large scale flooding due to extreme wave actions, catastrophic incidents, or bifurcation after reaching critical heeling angle needs to be considered.

This method of damage stability assessment will be based on the current methods for assessing the residual stability of the vessel and its likelihood of capsizing given the assumed damage. In this case, part of the reliability can be assessed through QRA, but this does not mean the actual failure criteria is identified. Determining a true capsize criteria for offshore structures is a challenging task. Approaches suggested by Vassalos [32], wherein a direct first principal based method is applied to determine the failure condition, which is then combined with a probabilistic method such as Monte Carlo simulation, could be applicable for offshore structure stability analysis as well.

With regard to damage stability of ships, International Maritime Organization Res. A265 (1973) [33] introduced the probabilistic damage stability method for passenger ships. In 2005, IMO Resolution MSC.194 (80) [34] adopted the harmonized probabilistic damage stability regulation for both dry cargo ships and passenger ships. A similar concept has been used in the offshore industry as part of risk assessment for structures, equipment and systems. However, stability is typically not considered in the risk assessment for offshore floating structures.

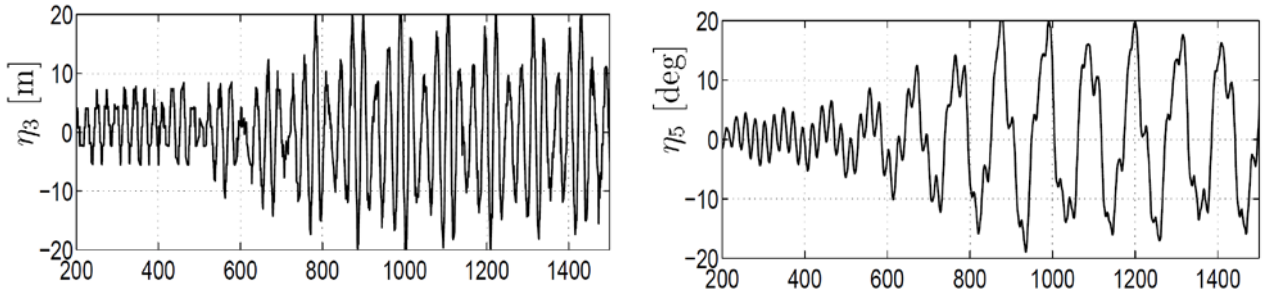


Figure 7. Parametric excitation in classic spar observed in model test by Haslum[25]

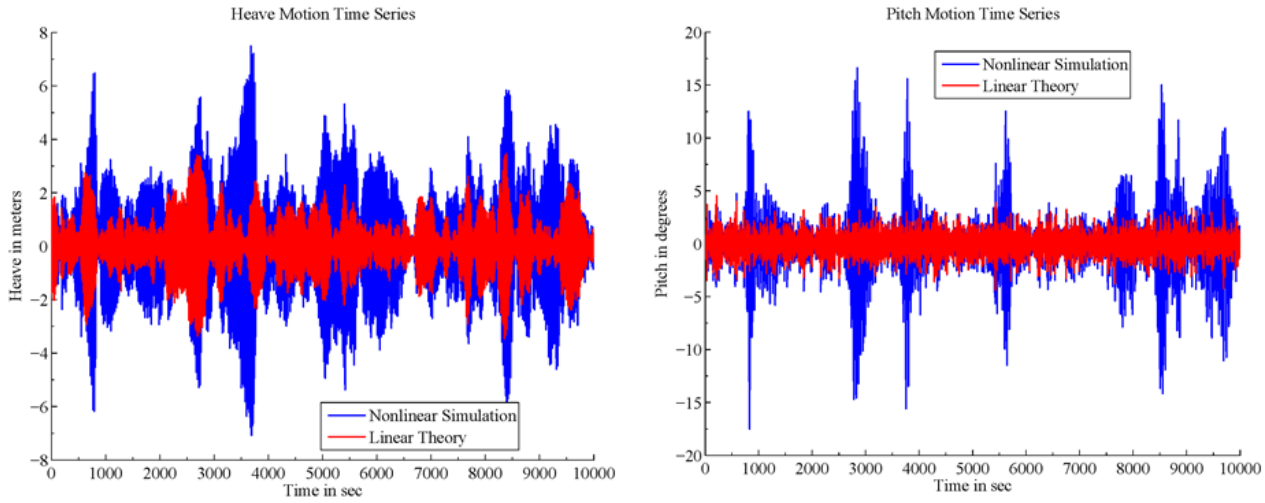


Figure 8. Parametric excitation in Single Column Floater[30]

6 CONCLUSION

Most of the stability/sinking failures that have occurred in the offshore oil and gas industry have not been due to divergence from existing standards, but due to other risks that are not part of the current stability calculation methodology. In almost all cases multiple failures were necessary for the results to be catastrophic. The stability criteria for offshore floating structure are traditionally based on a deterministic, quasi-static approach. Dynamic-response-based approach has been developed but is adopted at different paces by class societies, IMO, and local regulatory bodies. Although, dynamic instability is believed less an issue for offshore floating structure than for ships, there is a number of cases where parametric excitation presented detrimental motion of the platform.

Reliability methods are not suitable for the stability analysis based on quasi-static responses. The dynamic response-based stability analysis provides a superior method of quantifying uncertainties. However, without a clear definition of the failure mechanism and the limit state for stability, the reliability analysis appears to have difficulties in producing meaningful results.

The risk-based approach is gaining acceptance for damage stability assessment for certain types of ships. For offshore floating structures, more industry awareness should be encouraged.

7 RECOMMENDATIONS FOR FUTURE STUDY

Risk Assessments should take a greater interest in potential events that could cause a stability/sinking impairment. Because risk analyses are normally carried out anyway, better advantage should be taken of the process to address stability issues as well.

Circumstances that will lead to actual stability failures need to be identified first so that a stability failure criterion can be established. Only then can the reliability assessment be conducted in order to determine the probability of capsizing. Combining the probability of intact capsizing and the probability of damaged capsizing weighted by probabilities of damage events, one can determine an overall probability of capsizing failure.

In Summary, existing stability calculations and criteria should remain unchanged until there is a better understanding of the actual failure causes/mechanisms, and how to calculate for them.

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12.2 Presentation by Dag Erling Engberg

NTNU Norwegian University of Science and Technology

International Association of Oil & Gas Producers

Reliability of floating platforms with respect to stability

Dag Erling Engberg/DNV GL

The 3rd Offshore Structural Reliability Conference
OSRC2016
14-16 September, Stavanger, Norway

Stability related incidents

- Worldwide Offshore Accident Databank (WOAD)
- https://www.dnvgl.com/services/world-offshore-accident-database-woad-1747

- The fair share related to:
- Ballasting operations
- Loss of watertight integrity

Agenda

- Design considerations
- Operational considerations
- Competence, awareness and training

- Focusing on column-stabilized units

Potential reliability issues

- Design:
  - Flooding of buoyant volume above deepest waterline not being considered/ accounted for
  - Watertight integrity dependent on human action (openings in watertight boundaries)

- Operational:
  - Inadequate weight control
  - Loss of watertight Integrity
  - Inadequate operational competence and awareness

Design considerations – Flooding of deckbox volumes

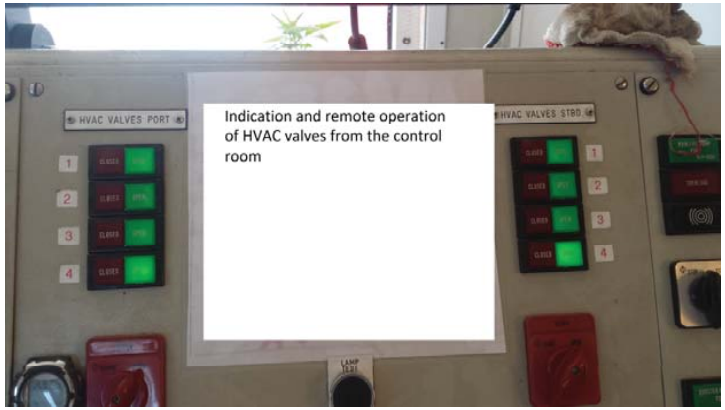
- Flooding of spaces above the maximum operational draught
  → Not covered by stability rules/regulations
- Flooding of deckbox volumes – Semis
  - Pressurised systems: Carefully consider routing of sea water piping. Main fire line, additional pumps automatically starting upon pressure drop
  - Fire water capacity significantly higher on production units than on drilling units
  - Large deckbox voids → small leaks could over time develop into a major threath
  - Fire fighting could lead to flooding of deckbox volumes
  - Weight of additional water & effect of free surface having negative impact on the stability
- Watertight subdivision, detection & drain

Design considerations – Openings in watertight boundaries

- Watertight integrity dependent on human action:
  - Doors, hatches, valves
  - Normally closed, normally kept open
- Closing appliances normally kept open at sea should be limited or rather avoided
- Example on next slides

HVAC openings and valves

HVAC valves – Remote operation and indication



HVAC Valves in watertight bulkheads

- Indication, function test and leak test

Watertight Integrity

- Breach of the watertight structure (hull, decks & bulkheads) from e.g. corrosion, collision, cracks
- Watertight closing appliances being kept open: Doors, hatches, HVAC
- Are we confident that someone will remember to manually close them in an emergency situation?

Operational considerations – Weight control

- Fixed load: Lightship weight
  - Inclining Test at NB (lightship weight and CoG)
  - All weight changes afterwards to be logged and accounted for in daily operation by the on-board crew (operational MODU requirement)
- Variable load: Deadweight, VDL
  - Constant monitoring of variable load, regular deck surveys and daily stability calculations by the on-board crew (operational MODU requirement)
- The critical parameter for stability being the Vertical Centre of Gravity
- 2009 MODU Code: Responsibility shifted further from Flag/Class to Rig Owner (less verification)

Competence and awareness – Damage response strategy

- Initiative from the Cruise Line Industry that could be relevant for floating platforms.

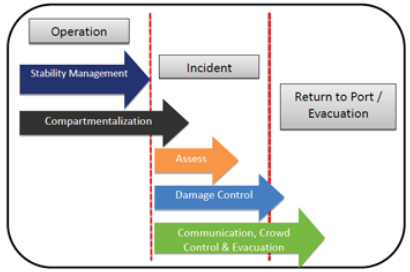


Figure 1: Damage response strategy

Damage Control Drills

- IMO SDC 2/17/1 proposes a new SOLAS regulation II-1/19-1 to require monthly damage control drills on passenger ships
- Increased focus and awareness through training of handling emergency situations, improved assessment of the situation, reducing the risk of human errors potentially escalating the situation (compensating measures going wrong).

Summary

- Deckbox volumes should also be checked for flooding scenarios (pipe rupture, leak, fire fighting). Pipe routing, watertight subdivision, leak detection and draining to be considered.
- Openings in watertight boundaries should be kept closed at sea.
- Accurate weight control to be a priority.
- Increasing competence and awareness through e.g. damage control drills is suggested.

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Chapter 13

Session 11: Reliability of Floating Platforms

13.1 Presentation by Jim Stear

NTNU Norwegian University of Science and Technology

International Association of Oil & Gas Producers

Industry Standards for Integrity Management of Floating Systems

James Stear / Chevron

The 3rd Offshore Structural Reliability Conference
OSRC2016
14-16 September, Stavanger, Norway

Review - Aging Floaters

- Continued service becoming a critical issue for the US Gulf of Mexico:
  - 11 platforms have been in service 15+ years
  - 16 more for 10+ years
- Many have riser / mooring system components at or close to original design lives
- **Need standardized process for evaluating continued service of floating systems**

Overview

- Aging Floaters
- Offshore SIM - Practice
- Floating System IM
- Path Forward

Previous Experience – 2SIM

- We've seen this issue before – fixed Gulf of Mexico infrastructure, 1950s on (1000s of structures by 1980s)
- RP 2A Section 17 outlined process in early 1990s
- RP 2SIM formalized it in 2014
  - SIM process
  - Continued service
  - Performance levels
- Now working 19901-9…


Structural Integrity Management of Fixed Offshore Structures

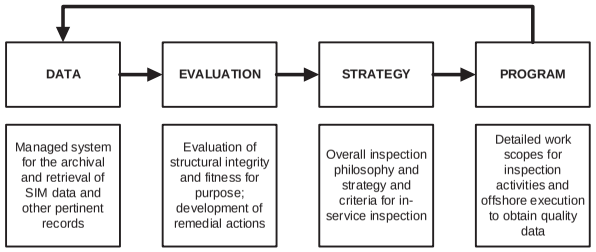
API RECOMMENDED PRACTICE 2SIM
FIRST EDITION, NOVEMBER 2014

API

2SIM – The Process

- Structural Integrity Management (SIM):

"an ongoing process for ensuring the continuing fitness-for-purpose of an offshore structure or fleet of structures".

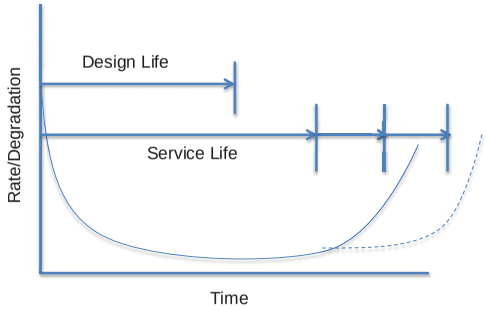


Fixed vs. Floating SIM: Major Differences

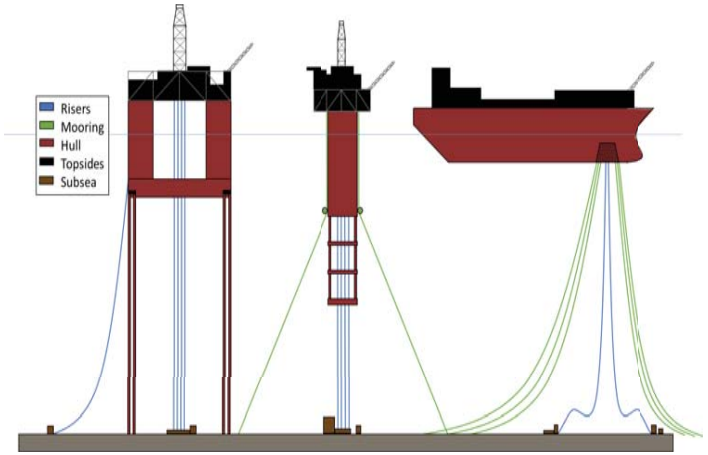
- Fixed: focus is often assessing yield / collapse limit states under storm wave loads
- Floating: multiple load types (storm winds / waves, currents), different component limit states, many definitions of failure, component fatigue as critical as overload
- BUT: process steps are the same

2SIM: Continued Service / Life Extension

- Philosophy is "Continued Service"
- Design Life ≠ Service Life (SL often ≥ DL)
- Continued Service = incremental steps in increasing service life
- Life Extension = sum of the incremental steps



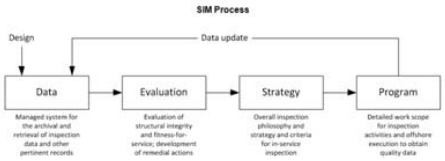
Floater IM Interfaces

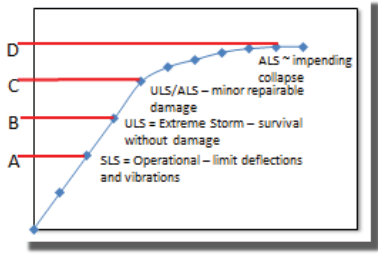


Systems-level view

Floater IM Gaps Identified Q4 2015

- No specific riser guidance
- Need for consistency of separate IM efforts with existing SIM process
- No integrated system-level performance criteria
  - Consistency with 2GEN philosophy
  - Overlapping hull / mooring / riser limit states
  - Fatigue limits – importance of single event
- Alignment at overlap points
  - FSIM / MIM / RIM / SC17 / SC6

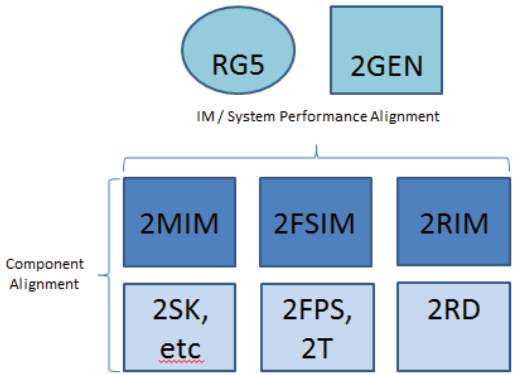




Mapping A-D to system-level targets for a floater…?

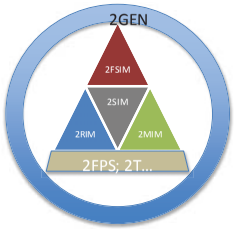
Vision for Floater IM RPs:

- Aligned IM suite for floaters, delivering consistent system-level performance in continued service



FEAT – Floater Evaluation and Assessment Team – formed Q1 2016

- Task Groups:
  - 2FSIM, lead = Jack Kenney / Shell
  - 2MIM, lead = Bob Gordon / consultant
  - 2RIM, lead = Stephen Hodges / Shell
- Coordination / liaison:
  - FEAT lead = Sathish Balasubramanian, ExxonMobil
  - 2GEN liaison = Dave Wisch
  - RG5 / 2SIM liaison = Hugh Westlake



2FSIM Scope

INCLUDED:

- **All FPS forms**: hull structure, hull mechanical systems, deck structure, all structural appurtenances (e.g. riser baskets, umbilical pull tubes)
- **For Tension Leg Platforms (TLP's):** tendon porches, tendons, tendon foundations
- **For Spars and Semisubmersibles:** fairleaders, hawse pipes, chain jack foundation porches
- **For FPSO's:** turret

EXCLUDED:

- Risers
- Conventional Moorings
- Umbilicals
- Process Equipment

2MIM Scope

- Intended for Permanent Mooring Systems (FPSO, FSO, FPU, CALM, etc)
- New and Existing Moorings
- Load Bearing Components of Mooring Anchor to Primary Steelwork
- Extends to Requisite Systems to extent required to manage Mooring

- Integrity:
  - Turret Bearings
  - Fairleads
  - Chain Stoppers
  - Thrusters (for Thruster-Assisted Moorings only)
  - Mooring risk control measures
- Not intended for:
  - Drilling Moorings
  - DP Moored Vessels without physical mooring hardware connection to the sea floor
  - TLP Tendons

Existing vs. New Build

Fixed GOM Platforms, 2SIM:

Category	API 2A Design Edition / Return Period		
	19th and Earlier	20th or 21st	22nd and Later
L-1	300-yr FPH*	300-yr FPH*	1,000-yr FPH
S-2	2,500-yr SH	2,500-yr SH	500-yr FPH
C-2	25-yr FPH	300-yr FPH	500-yr FPH
L-3	10-yr FPH	100-yr FPH	100-yr FPH
Use higher of L-1 / S-2 criteria for 21st Edition and earlier			

Following 2GEN, for floaters:

- **Life safety / environment protection to latest standards**
- **Economic risk, allowable load reduction consistent with fixed platform philosophy / onshore community for existing vs. new**

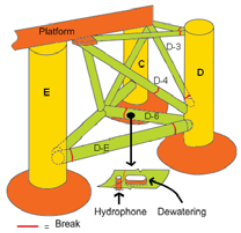
2RIM Scope

- All dynamic risers connected to <u>permanent</u> floating platforms (rigid, flexible, hybrid, TTR, drilling, etc)
- Umbilicals with hydrocarbons (i.e. gas lift)
- All riser components relevant to integrity of the riser, e.g.
  - Tensioners
  - Top connections – flexible joints, stress joints, flexing pull-tubes, etc
  - Corrosion protection
  - Buoyancy
  - VIV suppression

Does NOT address MODU drilling risers

Details, details…

- Fatigue checks / inspections key
- Assessment analyses for floaters / moorings / risers often skip key details, i.e. how connections / components actually behave
- "Minor" changes to structure / additions often missed
- *Any analysis must reflect reality not idealization*



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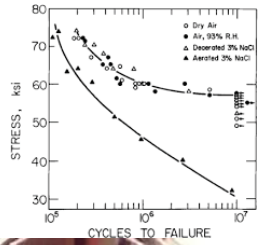
Some Concerns Identified…

- Fatigue design relationships
  - Identifying areas of potential relaxation from design margins
- Polymer components
  - High uncertainty in properties / longevity
- "Un-inspectables"
  - Examination of retired components
  - New inspection tech needs



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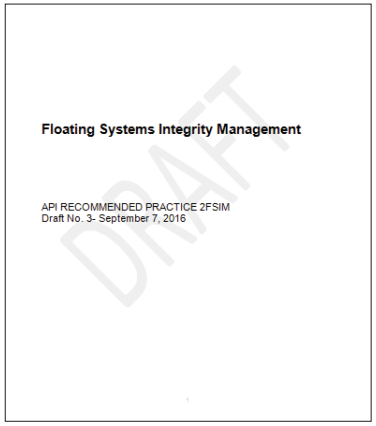
FEAT Progress

- DeepStar JIP draft FSIM documents donated to TGs Q1 2016
- TGs routinely meeting
- Engagement sessions:
  - Workshop with TGs and regulators held June 9
  - Workshop with OOC and regulators held August 31
- Drafts to be mostly done by Q1 2017
- **First editions to be published by end 2017**


Floating Systems Integrity Management

API RECOMMENDED PRACTICE 2FSIM
Draft No. 3- September 7, 2016

DRAFT

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Acknowledgements / Closing

- DeepStar 12401 TAC members (Anadarko, BG, BP, Chevron, Maersk, Woodside) and the Energo Project Team
  - The draft FSIM documents developed were a great kick-start for the API SC2 efforts
- API SC2 FEAT TGs
  - The energy shown in moving the new 2FSIM, 2MIM and 2RIM RPs forward is greatly appreciated
- BSEE / USCG
  - Comments / feedback have been extremely helpful for TGs

13.2 Presentation by Rolf Løken

NTNU Norwegian University of Science and Technology

International Association of Oil & Gas Producers

ALS design in practice - floating platform application

Rolf Løken/Aker Solutions

The 3rd Offshore Structural Reliability Conference
OSRC2016
14-16 September, Stavanger, Norway

Content

- Regional differences
- Mooring Systems
- Stability
- Air-gap

NORSOK N-003 Actions and actions combinations

Focus area for this presentation

	TEMPORARY CONDITIONS NOTE 1					NORMAL OPERATIONS				
	SLS	FLS	ULS	ALS Abnormal effect	ALS Damaged conditions	SLS	FLS	ULS	ALS Abnormal effect	ALS Damaged conditions
Permanent actions	EXPECTED VALUE (weight and buoyancy)					EXPECTED VALUE				
Variable functional actions	SPECIFIED VALUE	SPECIFIED ACTION HISTORY	SPECIFIED VALUE			SPECIFIED VALUE				
Metocean and earthquake actions	Based on statistical data	Expected action history	Based on statistical data	NOT APPLICABLE	Based on statistical data	Dependent on operational requirements	Expected action history	Annual probability of exceedance $=10^{-2}$	Annual probability of exceedance $=10^{-4}$	Annual probability of exceedance $=10^{-2}$ NOTE 2
Weather restricted metocean actions	SPECIFIED VALUE	SPECIFIED ACTION HISTORY	SPECIFIED VALUE	NOT APPLICABLE	SPECIFIED VALUE	NOT APPLICABLE				
Deformation actions	SPECIFIED VALUE	Expected action history	Expected extreme value	SPECIFIED VALUE		EXPECTED EXTREME VALUE				
Accidental actions	NOT APPLICABLE			SPECIFIED VALUE (depends on measures taken)	NOT APPLICABLE			Annual probability of exceedance $= 10^{-2}$ NOTE 3	Annual probability of exceedance $= 10^{-4}$	Not applicable

Regional Differences

ALS impact on production semis in different regions

Gulf of Mexico

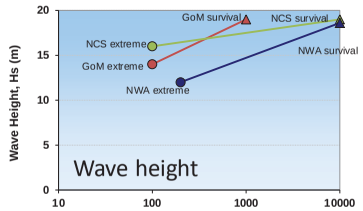
- Evacuated during hurricanes
- Robustness check for survival 1,000-yr hurricane typically not a major design driver
- Air-gap may be driven by 1,000-yr dependent on Client

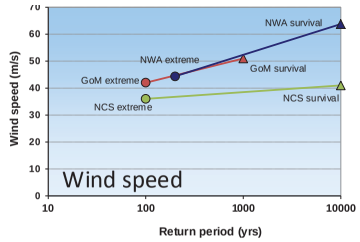
Norwegian Continental Shelf

- Multiple line failure scenarios for mooring is a driver for number of mooring lines
- Air-gap governed by 10^{-4} Waves for recent semi-submersible conceptual work on Vøring and Barents
- Large supply boats with ice reinforcement

North- Western Australia

- Manning philosophy dependent on distance to shore
- Severe cyclonic ALS conditions when manned
- In case of Ichthys semi-submersible, mooring, stability and air-gap driven by ALS and 10^{-4} wind and waves



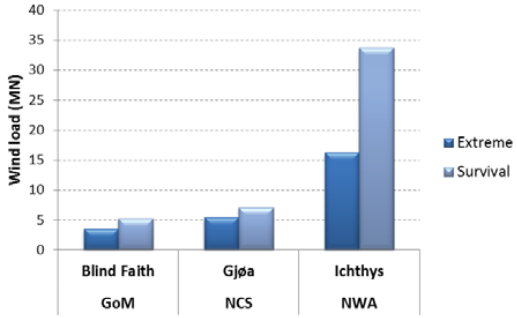


Regional Differences

Wind forces in perspective

10,000-yr static wind load

- GoM: 50% higher than ULS condition
- NCS: 30% higher than ULS condition
- NWA: 110% higher than ULS condition



Note: Bars represent different size semis and do not compare between locations

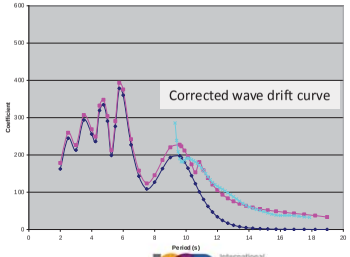
Mooring Line Failures

Status

- Frequency of line breakages (too) high
- Overload due to underestimation of wave drift force in design identified as one key issue

Wave Drift and Damping

- Historically included viscous correction in production floater design - model test based. Damping tuned accordingly.
- To a lesser degree applied for MODUs
- For MODUs and DP systems conservative damping assumptions may compensate for underestimation of drift force
- Seen from a designers perspective: Empirical formulation challenging – testing still required for proper correction



Mooring System Design Drivers

Rules and Design Basis

Governing conditions NCS

- ULS intact mainly governing condition for mooring dimensions, tension limits and offset
- ALS broken line (1, 2 and 3 lines removed) determines a minimum number of lines
- ALS intact with 10^{-4} environment typically not a design driver

Ichthys Semi in North West Australia

- ALS intact with 10^{-4} a clear design driver due to extreme winds in NWA

Ichthys semi illustration
28 Lines with 178 mm chain

Stability

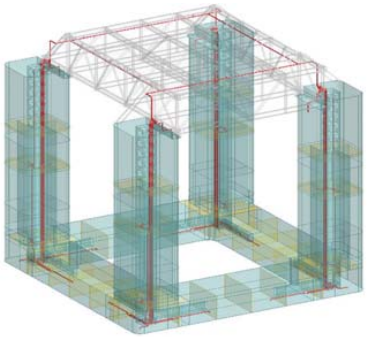
Barrier design for uncontrolled water movements

No hull penetrations

- No sea chests or through-hull connections
- Ballast water in/out through column top
- Limits possibility for inadvertent flooding of hull
- Pumps accessed externally only from column top

Four independent hull quadrants

- No bulkhead penetrations between hull quadrants
- No possibility for ballast water communication across hull quadrants



Hull Systems on Blind Faith 2005

Challenge

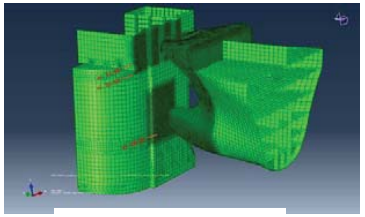
- Acceptance for Unmanned hull and reduced ballast system flexibility

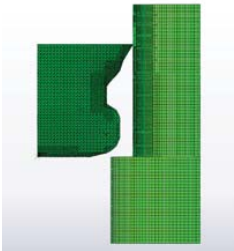
Stability

Damage Scenarios based on risk assessment

Ship collisions

- Larger and deeper supply boats → increased collision energy
- Increasing link between collision analyses and damage stability analyses
- Vessel type and speed typically given be standards or design basis
- Vessel approach angle and hit location chosen to be worst case scenario – not so risk based
- Collision risk still considered high – recent approach provide robust floaters





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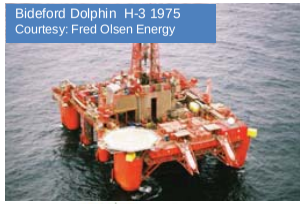
Air-gap on early rigs

Freeboard on the H-3's

- The H-3 has a survival freeboard of 17.3 m
- Based on experience from the ODECO rigs
- Model test 1972
  - Showed large clearance
  - Evaluated to be appropriate freeboard
- Air-gap analyses early 80's
  - Large air-gap margin
  - No diffraction or LF roll/pitch considerations
- Will still show positive air-gap under guidelines in draft DNVGL-OTG-13



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Air-gap analyses

- State-of-the-art analyses defined in draft DNVGL-OTG-13
  - Negative air-gap acceptable. Wave loading to be properly accounted for.
  - Analysis method in line with Aker Solutions's long standing practice for production and drilling semis (since mid 90's).
- Air-gap model test verification
  - Method extensively verified by model test
  - Generally good agreement with asymmetry factor of 1.2
  - Run-up impact loads from model test

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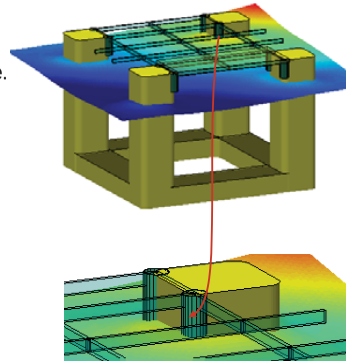
Air-gap requirement

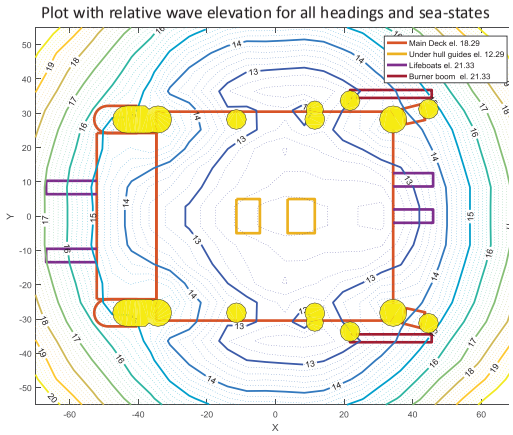
Freeboard typical for production semis

- ALS
  - Avoid water above lower deck on upstream side.
  - Particularly for open truss topsides which may lead to significant amount of water onboard
- ULS
  - Maintain positive margin of typically 1.5m in ULS/100-yr
- Predict local loads on underside of deck where freeboard is exceeded
- Strengthen underside of deck close to columns for run-up, possibly install wave deflectors to protect critical equipment



Air-gap results

- One plot to cover all headings and sea-states
- ➢ Information on positive air-gap
- If negative air-gap, several in-depth studies and plots required to predict actual extent and amount of water exceeding the deck level



Air-gap Results

If negative air-gap

- **If limited negative air-gap**
  - Local analyses with slamming loads
  - Analytical prediction is complex and time consuming, e.g. CFD
  - Possible to standardize based on sufficient model test info
- **If significant negative air-gap**
  - Clarify acceptance criteria both related to safety and global loads
  - Difficult to quantify and measure global loads
  - Wave in deck load guidance is available, mainly based on experience from tests with fixed platforms
  - Floater motions adds to complexity
  - How much is acceptable?

Summary

Practical ALS design

- In some parts a robustness check only
- In other parts an important design driver
- May be strongly non-linear or badly behaved and challenging to properly account for analytically
  - Extensive model tests may be required
- Increasing focus
- Important to consider in early stages of floater design

Thank You

Chapter 14

Session 12: Reliability of Ship type Production Units

14.1 Presentation by Erlend Hovland

NTNU Norwegian University of Science and Technology

International Association of Oil & Gas Producers

FPSOs in Harsh environment

Erlend Hovland, Jan Inge Dalane, Henning Mong /Statoil

The 3rd Offshore Structural Reliability Conference
OSRC2016
14-16 September, Stavanger, Norway

Content

- FPSOs ensure reliable production
- Hull design considerations
- Turret, mooring and riser considerations
- Offloading
- Life extension project findings

FPSO ensure reliable production

FPSOs/FSUs in operation
- Norne
- Åsgard A
- Åsgard C
- Peregrino
- Heidrun B
- Njord B
- Volve / Navion Saga

FPSOs/FSUs in construction
- Mariner B
- Gina Krogh FSO
- *Aasta Hansteen (Spar FPSO)*

Decomissioned FPSOs/ FSUs
- Lufeng / Navion Munin
- Glitne / Petrojarl 1

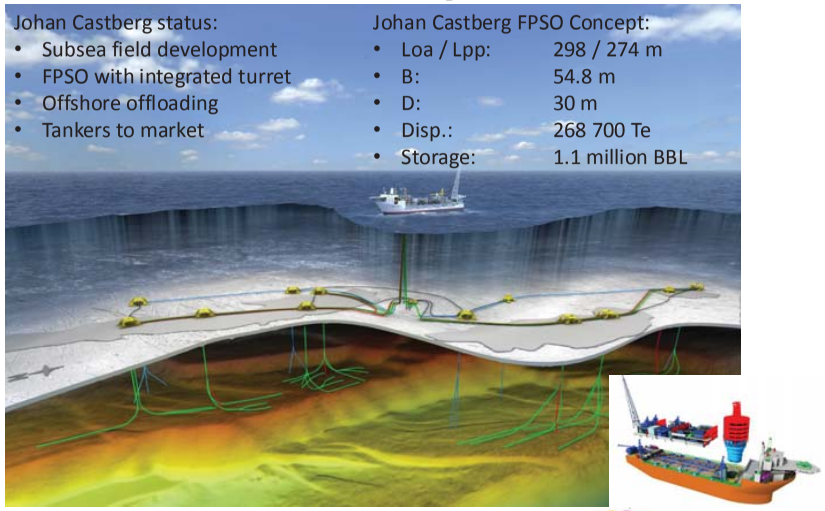
FPSO projects post DG1
- Johan Castberg

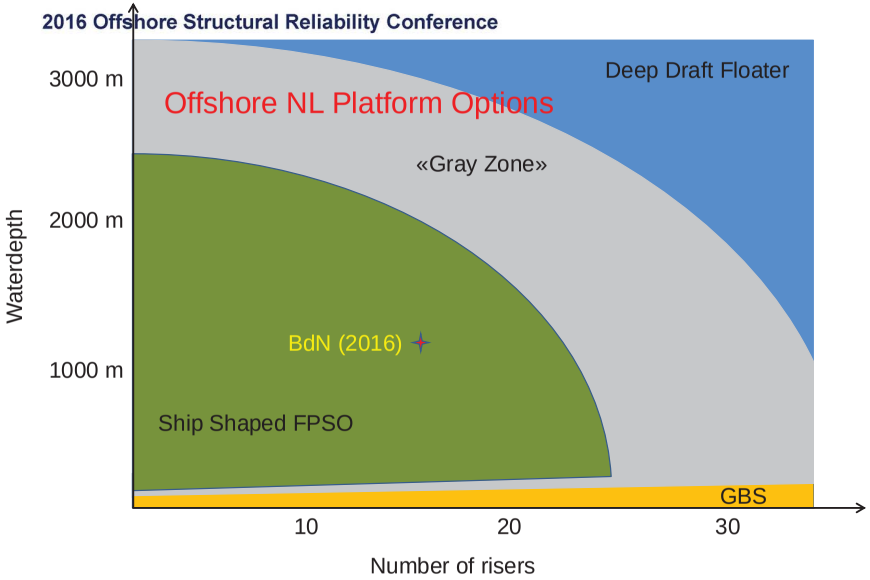
Early phase projects where FPSO is base case
- East Coast Canada (BdN)
- Brazil (BMC 33 ++)

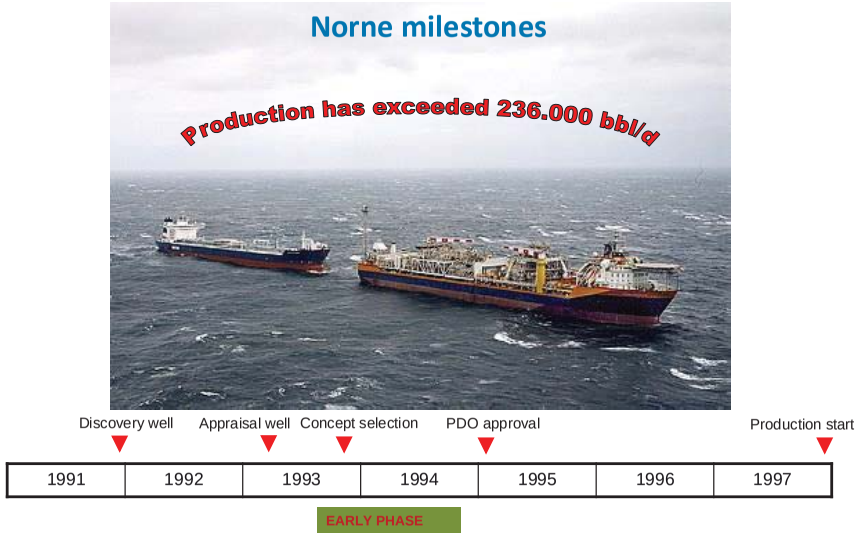
FPSO ensure reliable production

Johan Castberg status:
- Subsea field development
- FPSO with integrated turret
- Offshore offloading
- Tankers to market

Johan Castberg FPSO Concept:
- Loa / Lpp: 298 / 274 m
- B: 54.8 m
- D: 30 m
- Disp.: 268 700 Te
- Storage: 1.1 million BBL







Norne conclusions

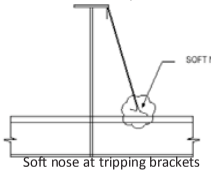
- Norne was developed on an extremely fast track schedule – a Statoil record.
- Norne was also the first Statoil operated FPSO project.
- Norne/Åsgard A has had their fair share of childhood diseases and challenges in operation.
- Considerable effort by highly skilled professionals.
- The original design was made adequately robust to allow for changes in design basis, modifications and upgrades in operation.
- Stabile production in extreme weather conditions.
- The concept has proven to be extremely successful (robust and flexible design).

Hull design consideration - Structure

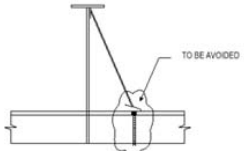
- Fatigue and cracking
  - Fatigue cracks in knuckle lines in fore and aft ship (interior) led to oil leaking from cargo to ballast tanks (first year of operation). Stiffeners added in cargo tanks solved the issue.
  - Side shell fatigue; low theoretical fatigue life of connection between longitudinal stiffeners and transversal frames. However, no cracks observed on Norne.
- Roll underestimated in design for fatigue and 100 year ULS
- Structural re-analysis with higher accelerations than originally designed for (handled by built in robustness).

Hull design considerations - Structure

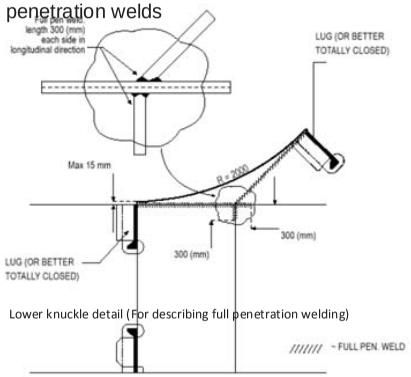
- Statoil developed FSU specification:
  - Particular attention to Fatigue (loads, load combinations, factors, calculation methods and critical areas)
  - Detailed NDT requirements
  - Structural details – does and do not's (experience based)
  - Strict requirements to usage of full penetration welds



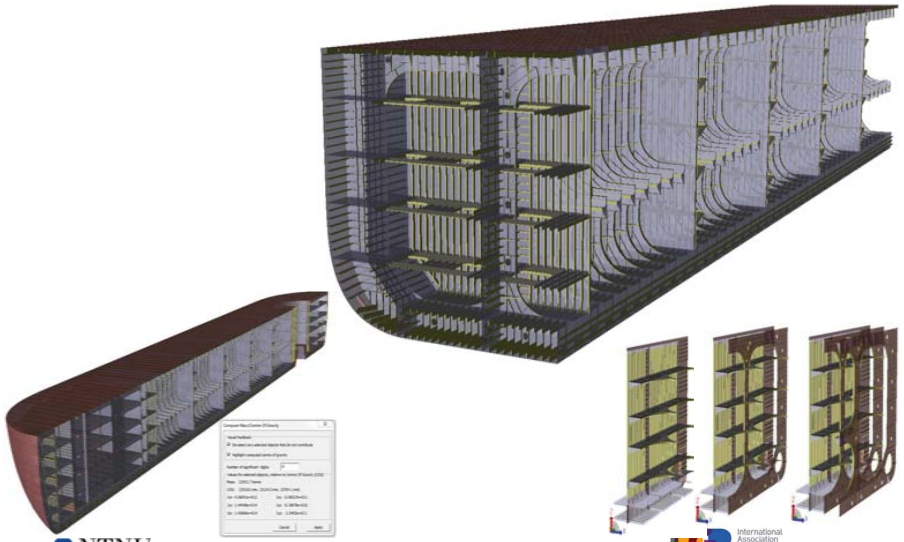
Soft nose at tripping brackets



Bad bracket end detail

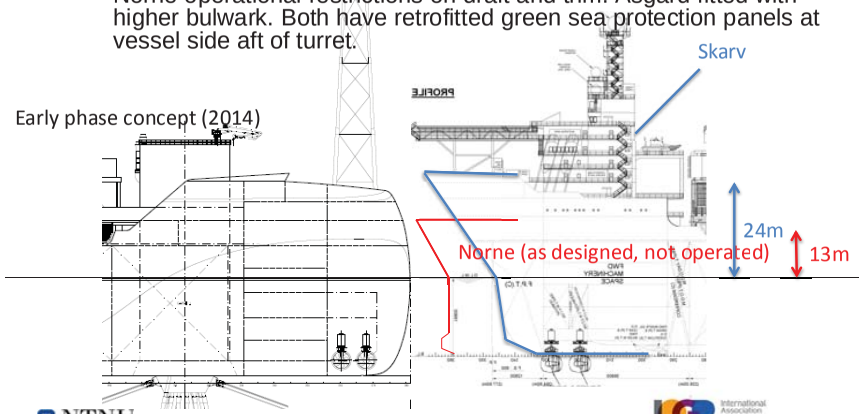


Lower knuckle detail (For describing full penetration welding)



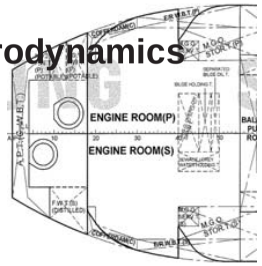
Hull design considerations - Hydrodynamics

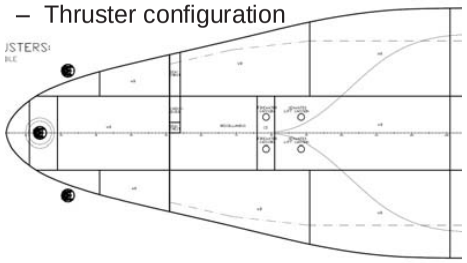
- Correlation between wind sea and swell. Effect of swell (non co-linearity) underestimated
- Green sea / operational limitations
  - Norne operational restrictions on draft and trim. Åsgard fitted with higher bulwark. Both have retrofitted green sea protection panels at vessel side aft of turret.

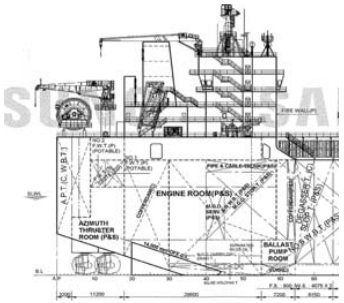


Hull design considerations - Hydrodynamics

- Passive weather vaning / heading control
  - Site specific metocean conditions
  - Turret position (Typ. < 25% of Lpp from FP)
  - Windage area and distribution
  - Mooring system and water depth
  - *Hull lines, in particular in the stern*
  - Thruster configuration



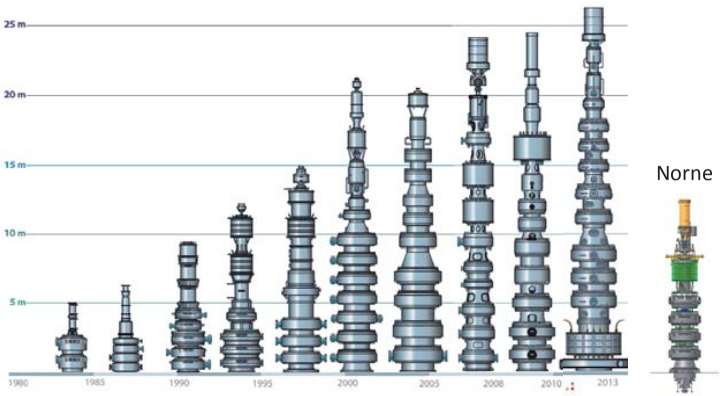




Turret and swivel

- Norne: 24 slots – 12 utilized at start up, 23 occupied today
- SBM swivels:
  - Generally worked well
  - Both Åsgard and Norne changed (in situ) mainly due to uncertain bearing status
  - Swivels modified to include high voltage capabilities
- Turret bearing maintenance is labour intensive (back log)
- Turret does not have «free rotation» due to friction. Turret drive and lock system modified and operational procedures changed.

Turret and swivel



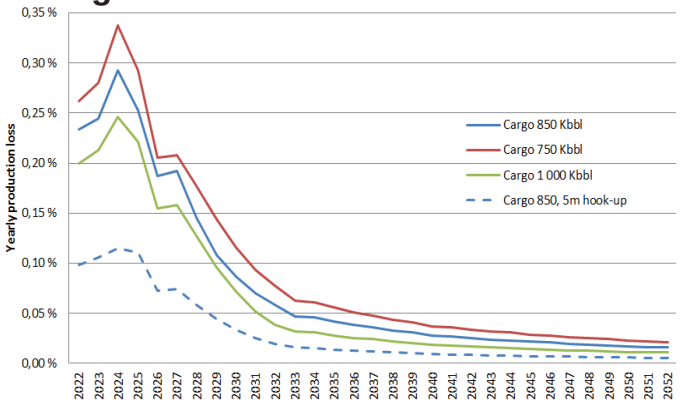
Risers

- Project phase (Norne)
  - High focus and extensive design basis document with robust load case matrix
  - Qualification testing performed
  - Extensive re-analysis within 5 years for Norne (basis for PUD approval)
  - Installation: One riser outer sheath damage for Åsgard & Norne combined! Possibly installation damage to Norne static WI pipe
- A lot of experience gained over the past 20 years on the subject of flexible risers (failure modes etc.)
- Some incidents & new developments in operation (ex. hydrate damage, singing, sliding buoyancy, water ingress in annulus)

Mooring

- Norne:
  - 12 mooring lines with 30 degrees spread angle:
  - Short fairleads with chain in contact with side resulting in out-of-plane bending if not optimal turret heading. Potential cause of line breakage on Norne in 2012.
  - Top chain changed due to MIC/SRB in guide tube (Microbe Initiated Corrosion/Sulphate Reducing Bacteria)
  - Fatigue analysis in design only for head sea (reduced life of top chain)
  - Top chain easy to change out
- Mooring component failure mainly a MODU issue
- Design is mainly challenging in shallow water (<120m) and in Deep water (>1000m)

Offloading



Shuttle tanker cargo	Average wating times [h]	
	FPSO: 1100 Kbbl	FPSO: 1400 Kbbl
750 Kbbl	21	1
850 Kbbl	29	5
1000 Kbbl	41	14

BdN shipping analysis, Sigurd Aanestad

Life Time extension

- Technical solutions proposed/preferred by fabrication yards e.g. in Asia (''Yard standard'') may not necessarily be optimal for Company in a long term perspective (Life Time Extension, Remote Location, Harsh environments, Limited Bed capacity, no Dry docking etc.)

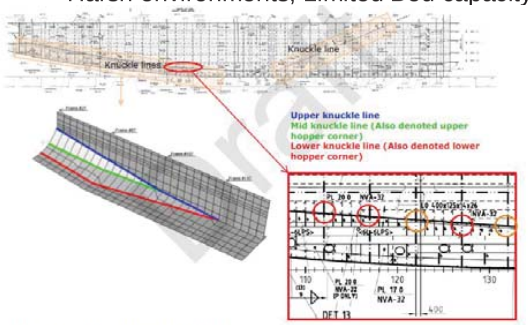
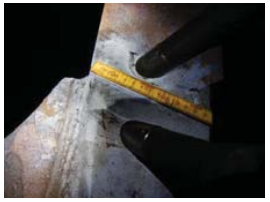


Figure 2-1 2012/2013 Inspection findings



Life Time Extension

- Norne:
  - Base case at start of Life Time Extension studies was dry docking in 2018
  - Extract from report dated 1 Nov 2014:

    The major challenge for lifetime extension of Norne FPSO was initially uncertainty related to the Technical Integrity of the hull. SRS Reanalysis /29/ and evaluation of inspection findings from 1997 to 2013 have been performed and it has been concluded, as a result of these activities, no need for docking in 2018.

  - Control of hull fatigue cracks
  - Good condition of paint and corrosion protection
  - Material selection in project phase
  - Inspection and maintenance
- Åsgard A:
  - One main focus in Life Time Extension studies in 2015 has been to evaluate the potential reduction in ULS and FLS capacity due to degraded paint and corrosion protection
- Overall conclusions for Norne and Åsgard A FPSO is that both can stay on location until 2030+ without disconnection and dry-docking
  - Scope for repair of fatigue cracks and repair of paint/corrosion protection is considered to be manageable within the available bed capacity

References

- https://www.norskoljeoggass.no/no/FPSO-Experience-Transfer/FPSO-Experience-Transfer/
- Svein-Arne Reinholdtsen, Statoil ASA
- Knut Harry Fjell, Statoil ASA
- Lars Gunnar Karlsen, Statoil ASA
- Narve Oma, PTIL (ex Statoil)
- Lars Geir Bjørheim, independent consultant (ex Statoil)

14.2 Presentation by Andre van der Stap

STRUCTURAL DESIGN BASIS OF WORLD'S FIRST FLNG TO ACHIEVE GOOD RELIABILITY

Andre van der Stap
TA1 Offshore Structures, Projects & Technology, SGS

Offshore Structural Reliability Conference, Stavanger
15 September 2016

A digital impression of Shell's floating LNG facility design

DEFINITIONS AND CAUTIONARY NOTE

Reserves: Our use of the term "reserves" in this presentation means SEC proved oil and gas reserves.
Resources: Our use of the term "resources" in this presentation includes quantities of oil and gas not yet classified as SEC proved oil and gas reserves. Resources are consistent with the Society of Petroleum Engineers 2P and 2C definitions.
Organic: Our use of the term Organic includes SEC proved oil and gas reserves excluding changes resulting from acquisitions, divestments and year-average pricing impact.
Resources plays: Our use of the term 'resources plays' refers to tight, shale and coal bed methane oil and gas acreage.

The companies in which Royal Dutch Shell plc directly and indirectly owns investments are separate entities. In this presentation "Shell", "Shell group" and "Royal Dutch Shell" are sometimes used for convenience where references are made to Royal Dutch Shell plc and its subsidiaries in general. Likewise, the words "we", "us" and "our" are also used to refer to subsidiaries in general or to those who work for them. These expressions are also used where no useful purpose is served by identifying the particular company or companies. "Subsidiaries", "Shell subsidiaries" and "Shell companies" as used in this presentation refer to companies in which Royal Dutch Shell either directly or indirectly has control. Companies over which Shell has joint control are generally referred to as "joint ventures" and companies over which Shell has significant influence but neither control nor joint control are referred to as "associates". The term "Shell interest" is used for convenience to indicate the direct and/or indirect ownership interest held by Shell in a venture, partnership or company, after exclusion of all third-party interest.

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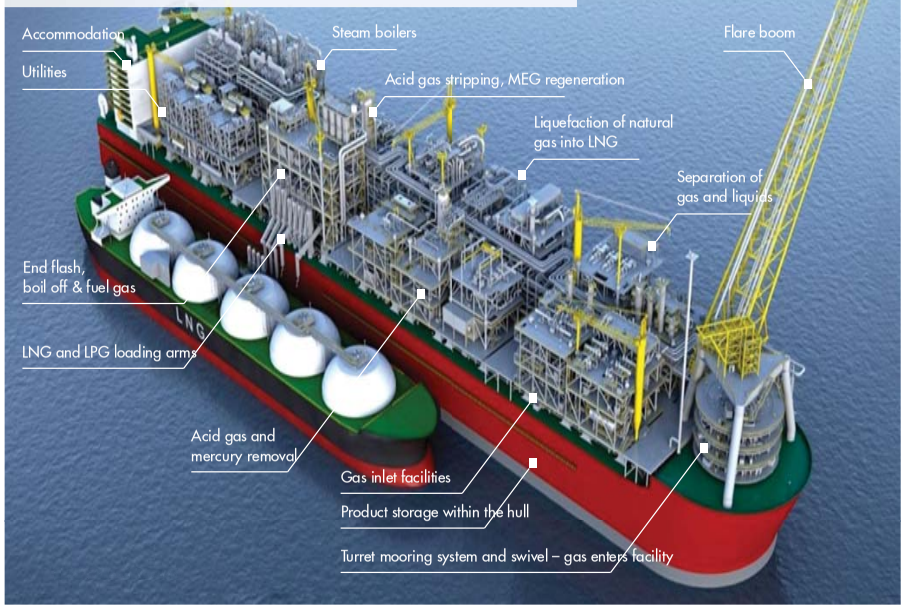
We use certain terms in this presentation, such as discovery potential, that the United States Securities and Exchange Commission (SEC) guidelines strictly prohibit us from including in filings with the SEC. U.S. Investors are urged to consider closely the disclosure in our Form 20-F, File No 1-32575, available on the SEC website www.sec.gov. You can also obtain this form from the SEC by calling 1-800-SEC-0330.

Agenda

- Introduction FLNG
- Design premise
- Design basis per component (typical examples)
- Design highlights (typical examples)
- Conclusions

PRELUDE FLNG FACILITY: LAYOUT

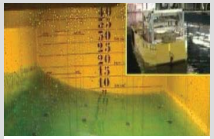


FLNG TECHNICAL CHALLENGES

- Liquefaction offshore, never done before
- LNG plant lay out on a limited plot size
- Mooring and offloading of LNG/LPG carriers adjacent to a (moving) production plant
- Possibility of extreme weather conditions

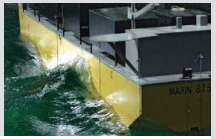
Results of our engineering:

- Safety on par with modern offshore facilities
- Availability on par with onshore LNG facilities


Sloshing test


Cryogenic spill protection test


Bridge simulation LNG carrier mooring procedure


Marine testing of extreme and normal conditions


Footprint


Loading arm test rig

DESIGN PREMISE

Overall Codes and Standards and their Hierarchy

- The FLNG structural design philosophy is aligned with two main standards: API and ISO
- Code and Standards hierarchy is as follows:
  - National Requirements
  - Project specific Basis for Verification
  - Shell Design and Engineering Practices (DEPs)
  - International Codes and Standards:
    - i) Industry Codes and Standards
    - ii) Classification Society Rules
  - International Guidelines of Engineering Practice

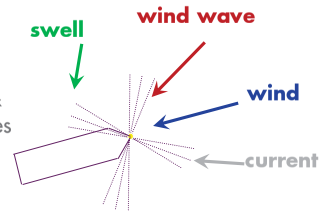
Design conditions according to ISO with typical action return periods

Limit State	Governing actions (to be designed for)	Action return period (typical)
Serviceability (SLS)	Resistance following from the criteria governing common operational use.	1 year
Ultimate (ULS)	a) Resistance following from maximum gravity actions.	100 years
	b) Resistance following from maximum environmental actions.	
Accidental (ALS)	Resistance following from situations of accidental or abnormal events.	10,000 years
Fatigue (FLS)	Resistance following from the accumulated effect of repetitive actions.	2-10 times service life

FLNG – STRUCTURAL DESIGN PREMISE

FLNG permanently connected & operated thru all weather conditions, incl cyclones

- Shutting down & disconnecting before cyclones would result in significant down-time (>20 days/yr)
- Shutdown of production as per Adverse Weather Policy (wind > 50m/s)
- Metocean Design Conditions: 100-year, 10,000 year
  - Hindcasting to obtain historical conditions e.g. for 50 yrs
  - Develop Joint Metocean conditions with return period of 100 yrs, & 10,000 yrs
  - Response-based approach: Convert metocean conditions to FLNG responses (e.g. vessel offset) & extrapolate each response to obtain extreme values (e.g. 10,000 yr load); backcalculate
- Model Tests



Magnitude, direction, timing

DESIGN BASIS PER COMPONENT (TYPICAL EXAMPLES)

Design conditions that are typically governing for the design of different structural systems

System	Operating 1 YRP	Extreme 100 YRP	Survival 10,000 YRP	Fatigue	Tow
Substructure			•		•
Mooring system		•	•		
Turret			•		
Topside structures	•		•	•	•
Production risers			•		
Water Intake Risers				•	

Metocean

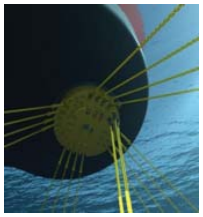
- Historical data collection
- Hindcasting and Synthetic storms database
- Joint extremes, response based

	Significant Wave Height, Hs (m)		
Return Period	PRELUDE	Central N Sea	Northern N Sea
1 yr	4.5	8.0	11.0
100 yr	11.6	13.0	16.0
10,000 yr	17.6	17.0	20.0

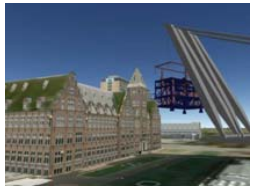
Conclusion: Extreme Cyclonic conditions at Prelude are comparable to extreme Central N Sea but less severe than Northern N Sea

PRELUDE FLNG TURRET & MOORING SYSTEM

- Mooring to provide station keeping
  - 4x4 groups (16 lines)
  - Chain-wire-chain, size 175mm, R4
  - Satisfies ISO 19901-4, plus survival in 10,000yr conditions
- 100yr conditions FoS=1.67
- 100yr condition with 1 line broken, FoS=1.25
- 10,000 yr condition, FoS=1.0



Topsides structural Design

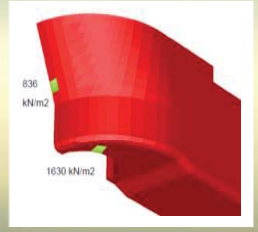


- Topside structures designed using API (AISC) WSD
  - Operating conditions with code allowables (e.g. $\sigma \leq 0.6\ \sigma y$)
  - Extreme with 1/3 increase
  - Survival with 2/3 increase
  - PFP and CSP applied based on fire, explosion and spill risk assessment

Load case	Description	Return Period	Allowable stress
Operating	Maximize dead & live loads	1	0.6
Extreme	Unit can be re-started after event with little no inspection & repair	100	0.8
Survival	Local damage does not lead to loss of integrity. No impact on people	10,000	1.0

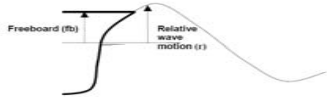
DESIGN HIGHLIGHTS (TYPICAL EXAMPLES)

RESEARCH AND TESTING AT MARIN



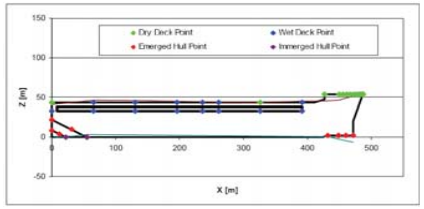
- Scale model 1:55
- Verify loads on mooring and turret during extreme weather
- Assess severity of conditions that are otherwise difficult to analyse
- Evaluate side-by-side offloading

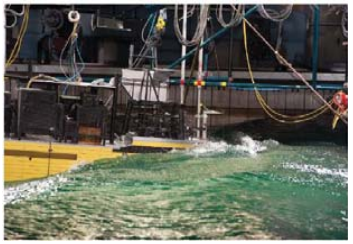
Green Water and Slamming 1



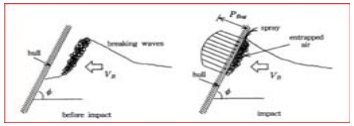
In Shell's Prelude occurrence of Green Water and Slamming was thoroughly assessed with both numerical CFD simulations and Marin scale model tests for a broad range of environmental conditions.

Under operating or 100yr conditions there is negligible risk of Green Water or Slamming. Some Green Water and Slamming has been observed under 10,000yr survival conditions, however compared to existing harsh environment FPSOs the susceptibility of FLNG hull to Green Water & Slamming is considered extremely low.



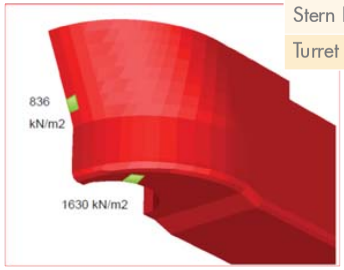


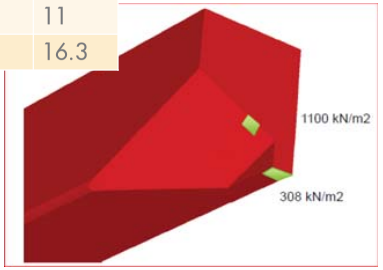
Green Water and Slamming 2



- Slamming depends on the instantaneous relative velocity and slope between wave and hull surface. Slamming pressure varies rapidly with time
- Maximum slamming loads on bow, stern and bottom
- Hull checked for following 10,000yr slamming loads ✓

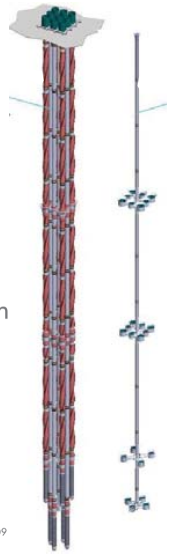
Location	MN/m²	Bar
Bow flare	0.836	8.4
Stern Flare	1.1	11
Turret underside	1.63	16.3





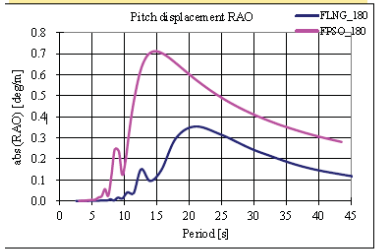
Water Intake Risers – Extreme, Survival & Fatigue

- Results for Extreme & Survival load cases
  - Von-Mises stress in whole riser is well below allowable stress ✓
  - Bending moment in Merlin connectors well below allowable moment ✓
  - MBR of rubber hose is very acceptable ✓
  - Potential for compression in Rubber Hose ✓
- Design of WIR dominated by fatigue: Most critical : weld between Merlin connector & pipe
- Wave-scatter diagrams with respect to vessel heading have been generated
- Fatigue life SF > 10 ✓

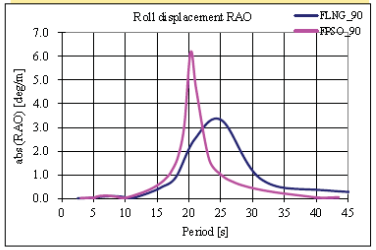


MOTIONS IN WAVES: FLNG v TYPICAL FPSO

Pitch Motion (Rotation forward)



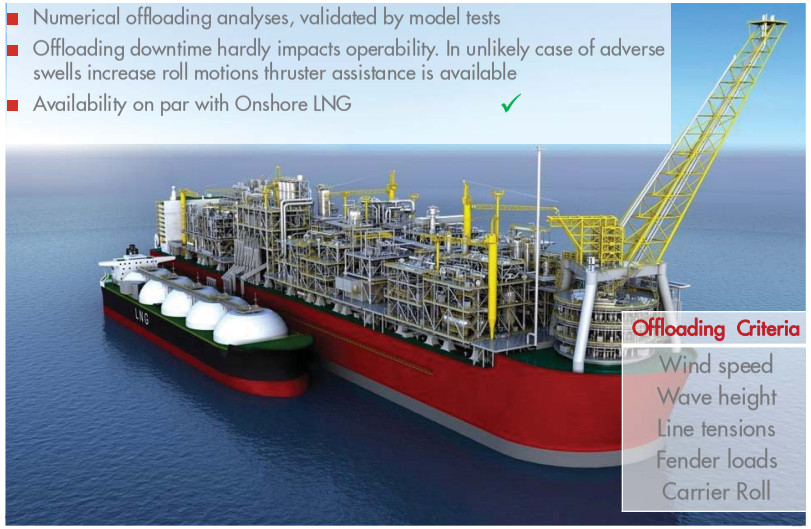
Roll Motion (Rotation sideways)



Conclusion: For any wave height and period all motions of FLNG are considerably lower than corresponding motion of an FPSO. Sometimes FLNG motions are 2-4 times lower !! Only exception is for roll motion in period range 23-35 sec. This range is of no practical significance since wave energy beyond 22sec is minimal. Roll response of FLNG is, in fact, excellent !!

FLNG: Side-By-Side Offloading and motions

- Numerical offloading analyses, validated by model tests
- Offloading downtime hardly impacts operability. In unlikely case of adverse swells increase roll motions thruster assistance is available
- Availability on par with Onshore LNG ✓



Conclusions

- Design and verification based on existing codes and standards and break down per structural (safety critical) components with consistent design targets has delivered transparancy.
- Systematic approach of 'state of the art' analytical, numerical and CFD analysis and verification with comprehensive model testing and analogies with existing facilities has given confidence in outcome
- Overall safety on par with modern offshore facilities
- Availability on par with onshore LNG facilities

Chapter 15

Session 13: Reliability of Station-keeping Systems

15.1 Presentation by Siril Okkenhaug

NTNU Norwegian University of Science and Technology

International Association of Oil & Gas Producers

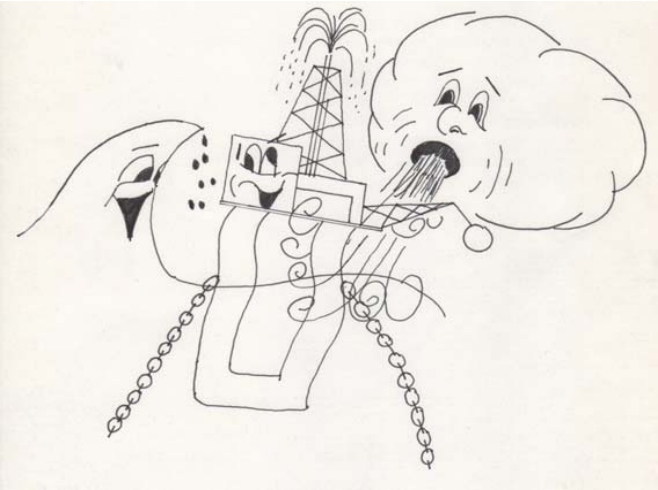
NorMoor JIP
– Mooring design code calibration

Siril Okkenhaug, DNV GL

The 3rd Offshore Structural Reliability Conference
OSRC2016
14-16 September, Stavanger, Norway

Contents

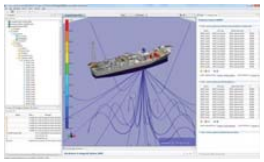
The Challenge



Mooring system integrity

Need to control the following phases:

- **Design** – robust w.r.t. loads and strength of mooring line components (ULS, ALS, FLS)
- **Fabrication** – with good quality
- **Installation** – good correspondence between installation procedures and design requirements
- **Operation** – good procedures in line with design requirements trained personnel
- **Condition monitoring** – maintenance, inspections, full scale measurements

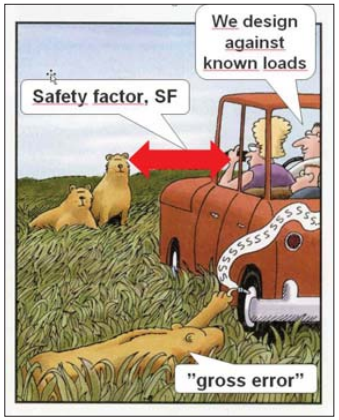


Current status – mooring integrity

- Too many incidents with loss of mooring lines

Year	Incident	Unit type	Single Failure	Failure freq. single line
2000-2014	PSA, Norway	mobile	30	11.5%
1997-2013	Deepstar, Worldwide	permanent	35	2.1%

- How can we improve?
  - We need to understand why the mooring line fails
  - So that our improvement actions will have an effect
- Phase of failure event:
  - Deepstar: 19% of failures related to design
    70% associated to installation & operation
  - PSA have similar numbers



NorMoor JIP – Background & Objective

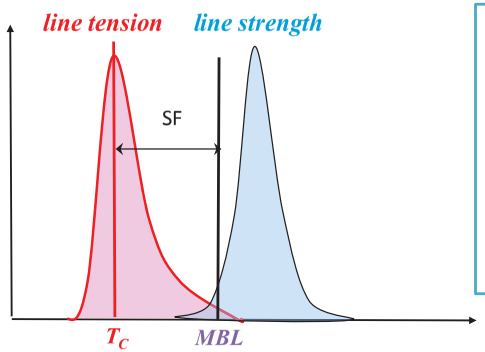
- To ensure mooring system integrity:
  - Need a safe design to start with!
- Current status:
  - Mooring standards are interpreted and applied in different ways
  - The safety level implied by the regulations is not known
  - The global standards (API, ISO, DNV, others) are mostly based on work from late 1990s when frequency domain analysis was prevalent
- Objective of the NorMoor JIP was to provide
  - a more consistent analysis methodology
  - calibrated safety factors for time domain
- NorMoor JIP – application
  - NorMoor JIP is a global study covering Northern Europe, GoM and Brazil waters, and is intended for global application
  - The aim is to get the results implemented into ISO-19901-7

Calibration of safety factors

- Mooring standards are based on simplified design procedures:

Rule criteria:
$$\frac{MBL}{T_C} \geq SF$$

MBL = minimum breaking load
T_C = characteristic mooring line tension, MPM based on worst short term condition with return period 100-100-10



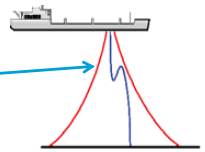
Calibration will be carried out for
1. "Ultimate limit state" (ULS)
   INTACT system
2. "Accidental Limit State" (ALS)
   DAMAGED system (line failure)
   In ISO this is a redundancy check in ULS with reduced SF

For both limit states SF's calculated for
CC1 Survival – unit disconnected
CC2 Operation – unit connected

How to calibrate safety factors

1. Line tension distribution for a range of units

   What is the tension in the mooring line?



2. Establish the annual probability of failure
   - How likely is it that a mooring line will fail?

3. Decide the target (consequence of failure)
   - Allow a larger probability of failure in Consequence class 1 (CC1) Survival (unit disconnected)
   - Stricter requirements in CC2, operation (production of hydro carbons)

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NorMoor JIP – Scope

- Geographical locations:
  - Haltenbanken
  - GOM
  - Brazil
- Water depths
  - 100-2100m depending on location
- Mooring systems applied:
  - Chain, chain+wire & chain+polyester
  - All designs are according to code requirements

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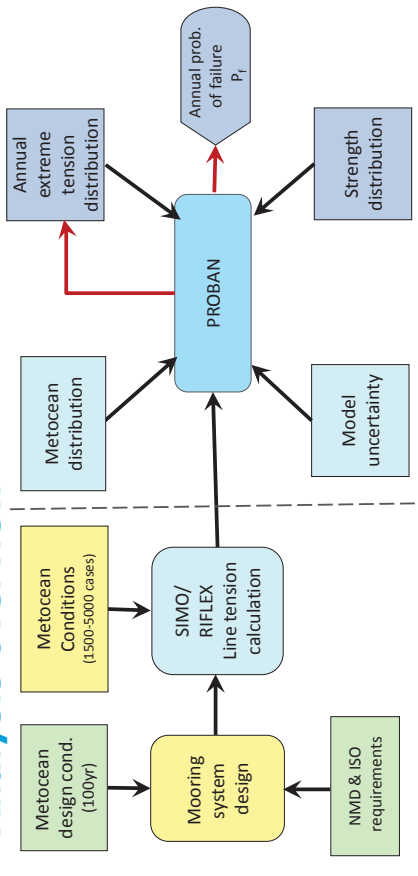
Analysis overview



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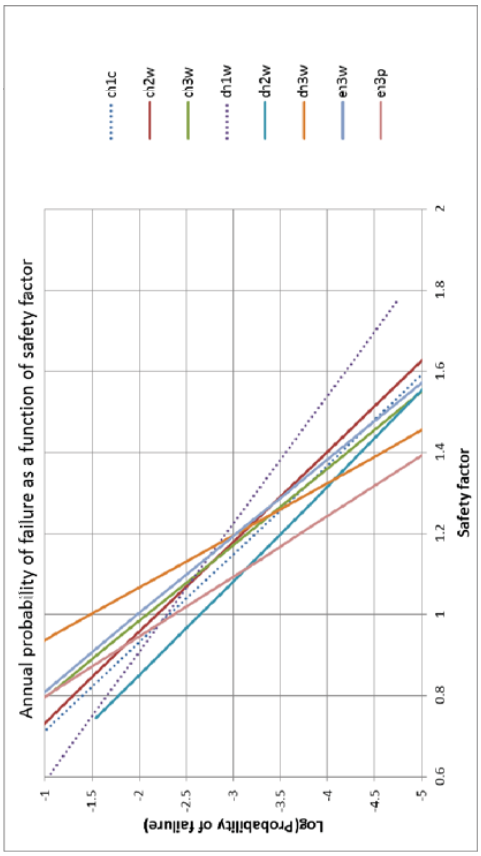
Annual probability of failure as a function of safety factor



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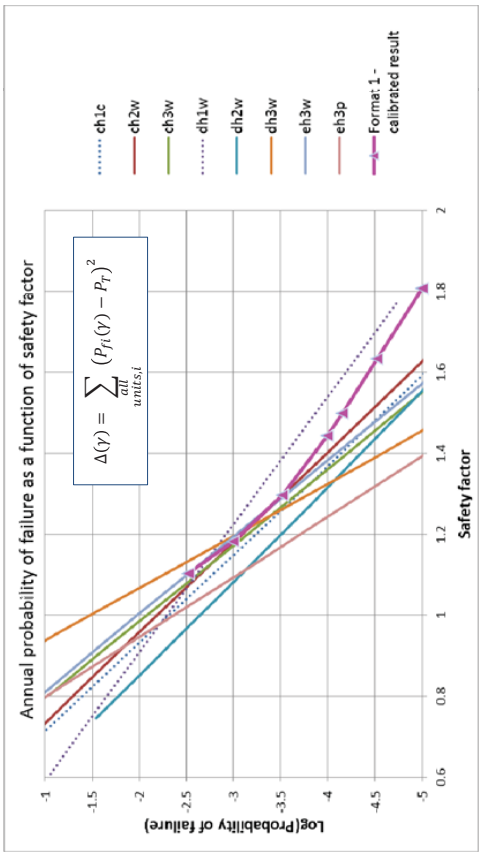
Calibrated result format 1 (MBL≥SF*T_C)



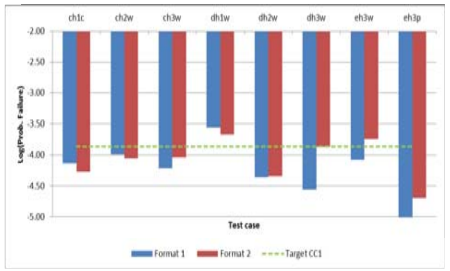
Alternative design formats

- Format 1 – one safety factor (ISO format)
  Format 2 – two safety factors
- Format 2 recommended by the NorMoor team
  - brings the test set closest to the target
  - reduced overdesign in deep water

1. MBL $= SF_{tot} * T_C$
2. MBL $= SF_p * T_p + SF_{env} * T_{env}$

T_C – characteristic tension
T_p - pretension
$T_{env} = T_C - T_p$

- Format 1 is used in this presentation for easy comparison with current practice (ISO format)

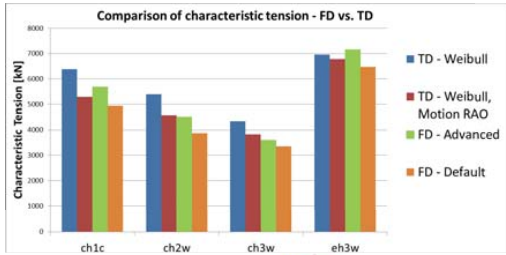


What about frequency domain (FD) analyses?

- FD calculations are still widely used especially for MOU's
  - Need a rule that also covers FD analyses
  - Can TD SF's be applied for FD analyses?
- FD results:
  - vary depending on analysis method
    - how line dynamics are implemented
    - choice of extreme value distribution (Rayleigh or more advanced distr.)
  - show significant variations compared to TD

FD give generally lower loads than TD

Unified requirements difficult



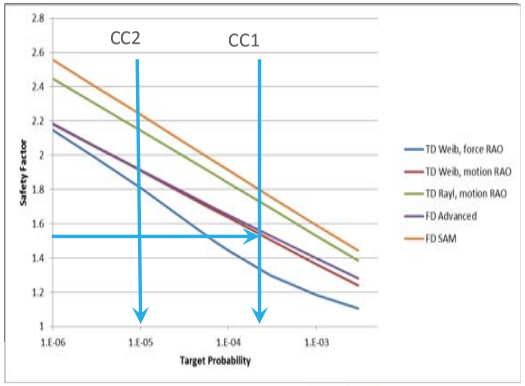
Target reliability from existing practice?

- Although the frequency of mooring line failure is too high, failure investigations point towards other causes than ULS/ALS design.
- ➔ We can select target reliability from existing practice (MOU CC1)
- What is existing practice?
  - Frequency domain (FD) analysis, NOT time domain (TD)
  - Use judgment FD results vary depending on analysis method

CC1: $P_{target} = 2 \cdot 10^{-4}$

- For CC2 (operation):
  - Keep the target from phase 1

CC2: $P_{target} = 10^{-5}$



ALS – one line failure

- ALS in the NorMoor project:
  - One line broken due to exceptional causes (weak line)
  - The probability of a second line failure is calculated (ordinary line)
- Most critical to have the second line failure during the same storm
  - no mitigatory actions can be taken
- Probabilistic formulation of combined failure event C:

$$P[C] = P[W \cap F_W \cap F_O] \approx \boxed{P[W] \cdot P[F_O]}$$

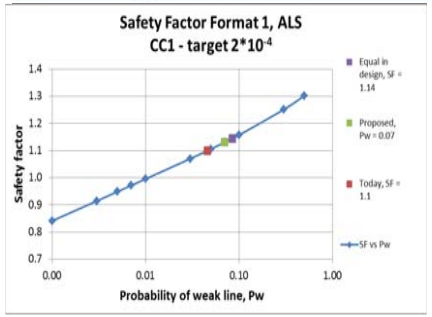
W – weak line
F_W – failure of weak line (first line)
F_O – failure of ordinary line (second line)

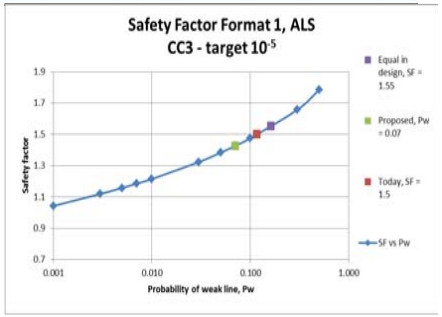
Probability of weak line P[W]

- P[W] is not known
- From the empirical study of mooring line failures (PSA and Deepstar data):
  - The annual probability that you have a weak line and that it fails
  - $r = P[W \cap F_W]$
    - Mobile: $r_{mob} \sim 0.014$
    - Permanent: $r_{perm} \sim 0.002$
  - There may be weak lines that have not yet failed.
  - Suggest to apply:
    - $P[W] = 5 \cdot r_{mob} \approx 0.07$
    - This is conservative for the permanent units, however ALS is not governing for these units

ALS results – time domain

	Safety factor ALS CC1 (survival)	Safety factor ALS CC3 (operation)
Current rules	1.1	1.5
DNV GL proposal	1.15	1.4
Equal in design, ALS≈ULS	1.14	1.55





- With reduced failure rates in the future

➔ the safety factors can be reduced

Recommendations from DNV GL's NorMoor team

- Safety factors and analysis methodology are interlinked.
- The delivery from the project is twofold:

1. Detailed guidance on how to do the mooring analyses in TD and FD
2. Calibrated safety factors (here limited to format 1 in tables below)
   - to be applied together with the above analysis guidance/methodology
   - format 2 with partial safety factors will be provided later

	ULS - intact – Format 1		Present rules
	Time domain analyses	Frequency domain analyses	ISO 19901-7 Annex B
CC1 survival, target~$2*10^{-4}$	1.4	1.5	1.5
CC3 operation, target~10^{-5}	1.9	2.2	2.2

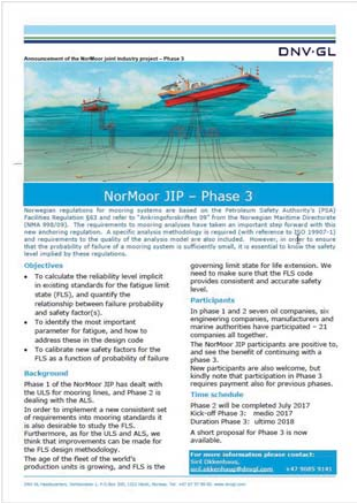
	ALS - one line missing – Format 1		Present rules
	Time domain analyses	Frequency domain analyses	ISO 19901-7 Annex B
CC1 survival, target~$2*10^{-4}$	1.15	1.3	1.1
CC3 operation, target~10^{-5}	1.4	1.6	1.5

NorMoor JIP - Status

- Phase 1 ULS (2011-2015) 9.4 MNOK
- Phase 2 ALS (2015-2017) 9.6 MNOK
- Participants Phase 1 and Phase 2:

Oil comp.	Other comp.	Manufacturers	Authorities
Statoil	DNV GL	Bexco	PSA
BP	APL NOV	Vicinay Cadenas (1)	NMD
GDF Suez	SBM		HSE
Det Norske	Transocean		
Total	Vryhof Anchors (1)		
BG (1)	Delmar (1)		
Petrobras			
Shell			

- Phase 3 FLS – aim for kick off medio 2017
  - Participants are positive
  - Will give us a consistent set of new requirements
  - FLS is the governing limit state for life extension
  - The aim of the new design rule is to:
    Remove unnecessary conservatism in the present practice & ensure that sub-standard designs are avoided.



15.2 Presentation & article by Haibo Chen

NTNU Norwegian University of Science and Technology

International Association of Oil & Gas Producers

Reliability of DP Systems

Dr. Haibo Chen / Lloyd's Register

The 3rd Offshore Structural Reliability Conference
OSRC2016
14-16 September, Stavanger, Norway

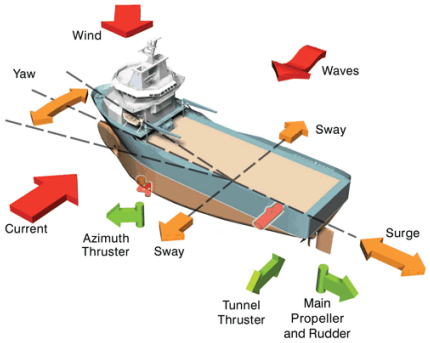
Basics: Reliability

- "Reliability" = Ability to perform a required function under given conditions for a given time interval.
- "Reliability" is also used as a measure of reliability performance and can also be defined as a probability.
- The "failure rate" is a classical reliability parameter traditionally denoted by the Greek letter, λ (lambda). The failure rate is an average frequency of failure (i.e. a number of failures per unit of time).

Reference: ISO 14224:2006, ISO 20815:2008

Basics: Dynamic Positioning

- Dynamic positioning (DP) is defined as to automatically maintain vessel's position, fixed location or predetermined track, exclusively by means of thruster force
- DP system consists of the following:
  - Power system
  - Thruster system
  - DP-control system



https://www.km.kongsberg.com/

Reference: IMO MSC/Circ.645, 1994

Reliability of DP systems

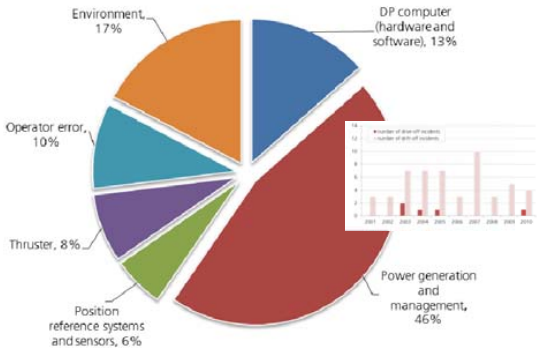
- IMO MSC/Circ.645 "Guidelines for vessels with dynamic positioning systems", 1994
  - Equipment class 1: loss of positon may occur in the event of a single fault.
  - Equipment class 2: loss of position is not to occur in the event of a single fault in any active component or system.
  - Equipment class 3: loss of position is not to occur in the event of a single fault in any active component, static component, and fire/flooding of any one watertight compartment or fire sub-division.
- Question: What is the "failure rate" for each class?
  - DP class 3 vessel = negligible likelihood of position loss?

Position loss modes

1. Drive-off
   - Abnormal thruster forces driving vessel away from its target position, and has an excursion beyond the acceptable excursion limit.
2. Drift-off
   - Insufficient thruster forces so that vessel is drifted away from the target position by the environmental loads, and has an excursion beyond the acceptable excursion limit.
3. Large excursion
   - A situation where the vessel is still under control of the DP system but has a temporary larger than average excursion of position or heading, because of large wind gust, wave, single thruster fault, worst case single failure, or degraded position information.

Example: Position Loss on DP MODUs in 2001-2010

- DP incidents on mobile offshore drilling units (MODUs) from the reported IMCA station-keeping incidents during 2001-2010.
- In total 57 position loss incidents on DP MODUs in this 10-year period, 5 drive-offs and 52 drift-offs.
- Causes to drift-off in the pie chart



(Chen and Nygård, 2016)

DP vessel position loss is not negligible.

- Sharp increase of the DP vessels worldwide (Bierman, 2014)
  - 65 in 1980
  - 150 in 1985
  - 2000 in 2011
- Wide applications of DP vessels in complex offshore marine operations, often in close proximity to offshore facilities.
- Collisions
  - PSA Norway: 26 collisions on the NCS during 2000-2010, many were DP vessels. (Kvitrud, 2011)
  - H&SE UK: ship/platform can be catastrophic. (Boothby, 2013)
- Major accident risk
  - Risk of damaging well integrity and pollutions
  - Risk towards divers
- Asset damage, reputation, etc. etc.

Example: Frequency of Position Loss on DP MODUs

- Under-reporting? = voluntary nature of station-keeping incident reporting => Assumption of under reporting factors.
- DP time relevant for the reported incidents? = unknown => Assumption of relevant DP MODUs and their operational time.
- Uncertain results: 559 DP years, in association with 15 drive-offs and 104 drift-offs.
- Position loss frequency: 0.21 per DP year related to DP MODUs in 10 years (combined class 2 & 3 vessels)

Year	Semisubs & Drillships	% for DP MODUs *	DP MODUs worldwide *	Utilization *	DP Operational Time (years)
2001	168	25%	42	80%	33.6
2002	168	25%	42	80%	33.6
2003	167	35%	58	85%	49.7
2004	166	35%	58	85%	49.4
2005	167	35%	58	85%	49.7
2006	169	35%	59	85%	50.3
2007	169	35%	59	85%	50.3
2008	178	45%	80	90%	72.1
2009	199	45%	90	90%	80.6
2010	221	45%	99	90%	89.5
				SUM	559

* Data based on Assumptions.

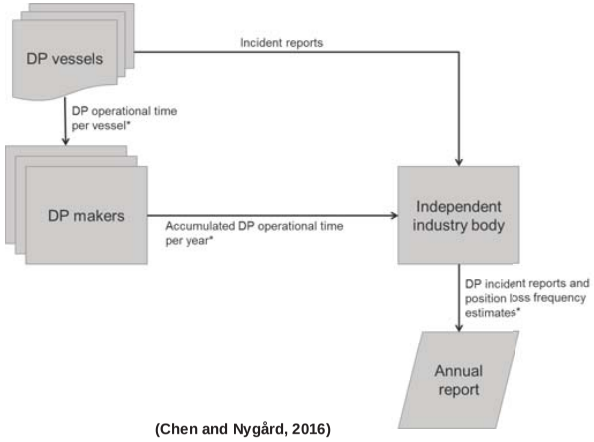
(Chen and Nygård, 2016)

Challenge: Frequency of Position Loss?

- The current DP incident reporting scheme = *Sharing* info of causes among DP vessels.
- How frequent did position loss events occur on DP vessels? = *No firm answer*!
- *The only thing for certain* = Position loss incidents did occur on DP2 and DP3 vessels in the past.
- *Future* = Even with modern DP technology and well-trained operators, such position loss will most likely *continue to happen*.
- *Complacency* = Tempting to neglect the position loss risk as long as a DP vessel is in compliance with the prescriptive DP2 and/or DP3 requirements (which were originated over 20 years ago!)
- Diminish DP position loss as a major accident hazard in the industry.

Proposal: A New DP Incident Reporting Scheme

1. The purpose is to combine incidents and DP operational time.
2. DP operational time can be collected automatically based on current technology from DP control system makers.
3. Benefits: More Reliable Position Loss Frequency Data



(Chen and Nygård, 2016)

DP reliability vs. risk in marine operations

Qualitative methods: Verification of Compliance

Traditionally, most qualitative methods addressing DP risk in marine operations are of "compliance verification" nature.

1. Ensure DP systems redundancy and verify compliance with the designed DP equipment redundancy class
2. Ensure DP operators meet the prescriptive qualification requirements
3. Ensure DP operations are performed and DP status are given according to the pre-defined criteria

RISK RANKING MATRIX		CONSEQUENCES CLASS				
		1. NEGLIGIBLE	2. DEGRADE	3. WORST CASE FAILURE	4. POSITION LOSS, MINOR CONSEQUENCE	5. POSITION LOSS, MAJOR CONSEQUENCE
FREQUECY CLASS	1. NEGLIGIBLE	1	2	3	4	5
	2. UNLIKELY	2	4	6	8	10
	3. POSSIBLE	3	6	9	12	15
	4. LIKELY	4	8	12	16	20
	5. PROBABLE	5	10	15	20	25

- FMEA, FMECA, and FMEA proving trials of DP system, independent survey
- Activity specific operating guidelines (ASOG)
- HAZID) and HAZOP of procedures
- Training and certification of DP operators

Question: Are these enough?
DP class 3 = No position loss risk?

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Quantitative method: Frequency Model

- Modern risk management practices and safety regulations in the offshore industry are built on a quantitative framework.

Frequency of position loss

P(accident) = P(position loss) ×

collisions, loss of well integrity, rupture of loading hose, etc.

P(failure of recovery)

- A practical formulation of DP Risk Model in Marine Operations.

Failure probability of recovery actions initiated by DP operator, or automatic safety systems, given position loss.

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Design inherent safe DP operations (2007-14)

DP shuttle tanker offloading from geo-stationary FPSO

1. Position with inherent safety
   - Weathervane with Heading pivot point
   - Min. 150 m no entry zone
2. Separation distance of 200 – 250 m
3. Special Hull structural design to protect mooring fairleads

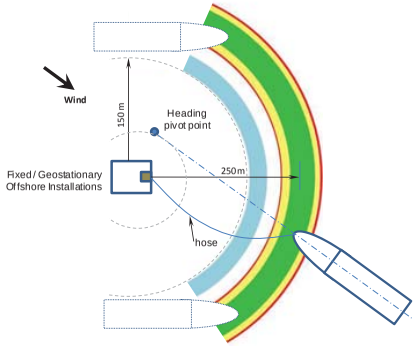

Illustration drawing: The size and distance are not to scale.

References:
Concept: Lerstad, Chen and Breivik, 2010
Collision risk: Chen, Lerstad and Moan, 2010

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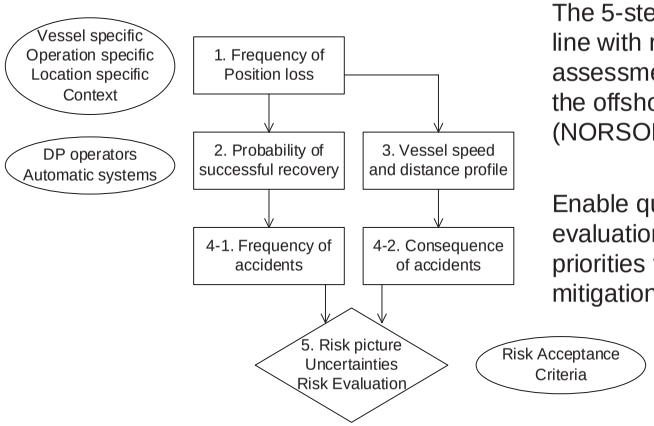
DP risk Quantification and Evaluation



The 5-step method is in line with risk assessment process in the offshore industry (NORSOK Z-013).

Enable quantitative risk evaluation, and provide priorities for risk mitigation.

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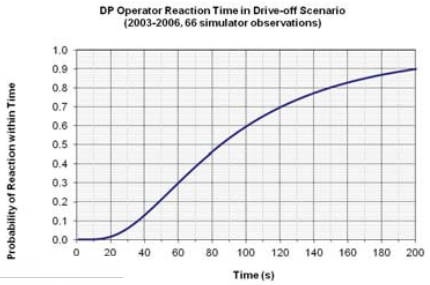
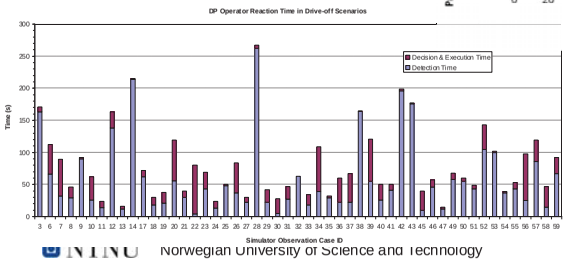
Human Reliability: DPO reacts to drive-off?

- Incident data, Expert judgment, 2001-2002
- DP captain questionnaire survey 2002
- Logging of DP operator reaction time during offloading simulator training (simulated drive-offs) in 2003-2006



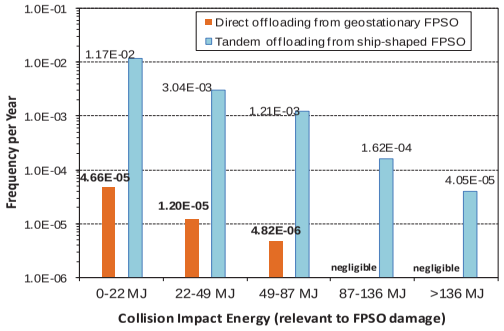


References:
Chen, Moan and Vinnem, 2007

Perform human reliability study to assess failure of DP operator given various position loss scenarios, e.g. SPAR-H method.

Collision Risk: Direct vs. Tandem Offloading

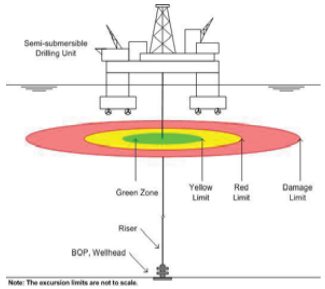
- The collision frequency in the direct offloading ($6.43 \cdot 10^{-5}$ per year) is much lower (less than 1%) than the equivalent tandem offloading from ship-shaped FPSO ($1.62 \cdot 10^{-2}$ per year).
- Premises
  - DP2 shuttle tanker
  - Offloading once per week
  - 24 hours operational time
- Mooring lines and risers are not subjected to shuttle tanker impact



Evaluate DP operational philosophies (2013-15)

CAT-D drilling unit in the Troll field

- The objective is to evaluate the risk of well integrity in the Troll field due to DP position loss.
- The focus is to analyse and compare risks given two different DP operational bases for the CAT-D rig: WSF2 vs. WSF1.
- Conclusion: WSF1 has equivalent well safety level as WSF2



(Chen, Sætre and Jahnsen, 2015)

WSF2
Worst case Single Failure of losing two (2) thrusters

WSF1
Worst case Single Failure of losing one (1) thruster

Conclusions & Recommendations

1. Reliability of DP system
   - DP equipment class 1, class 2 and class 3
   - Failure rate: Frequency of position loss is uncertain, but not negligible any more.
   - Reliability data: A new DP incident reporting scheme is proposed for reliable position loss frequency data.
2. DP reliability vs. risk in marine operations
   - Qualitative DP risk analysis: Methods and deficiencies
   - Quantitative DP risk analysis: A 5-step method is presented.
   - Cases and lessons:
     - Design inherent safe DP offloading operations
     - Evaluate DP operational philosophies in DP drilling operations.

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The 3rd Offshore Structural Reliability Conference
OSRC2016
14-16 September, Stavanger, Norway

Reliability of DP systems

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ABSTRACT: Reliability of DP systems is addressed in this paper. The IMO MSC/Circ.645 provides definition of DP system reliability in terms of three equipment classes, and prescribes single failures in relation to loss of position. This qualitative requirement of DP system reliability does not specify failure rate for each DP equipment class. It remains uncertain in the industry regarding what are expected frequencies of position loss for DP vessels, in particular for DP class 2 and class 3 vessels. Quantitative measure of the DP system reliability is needed in terms of position loss frequency. The paper describes the practices of deriving position loss frequency from historical DP incidents and the challenges involved. To provide transparent and consistent historical frequency of position loss, an alternative DP incident data reporting scheme is recommended. It combines both incidents and corresponding DP operational time. The reliability of DP systems is an essential input for managing DP risk in offshore marine operations. Traditional qualitative risk analysis methods and their limitations are discussed. This paper presents a quantitative approach for DP risk analysis and evaluation. It is in line with the risk assessment process in the offshore industry, e.g. NORSOK Z-013. Two case studies, i.e. DP shuttle tanker offloading operation, and DP drilling operation, are referenced to illustrate the methodology and benefits.

KEY WORDS: Dynamic Positioning, Reliability, Risk, Marine Operations, Quantification

1 INTRODUCTION

Dynamic positioning (DP) is defined in IMO MSC/Circ.645 as to automatically maintain vessel's position, fixed location or predetermined track, exclusively by means of thruster force [1]. A DP system consists of the complete installation necessary for dynamically positioning a vessel. It generally includes three main subsystems, i.e. power system, thruster system and DP control system.

The term reliability is defined in the ISO20815 [2] and ISO14224 [3] as follows. The reliability means the ability to perform a required function under given conditions for a given time interval. The term reliability is also used as a measure of reliability performance and can also be defined as a probability. In this context the failure rate is a classical reliability parameter traditionally denoted by the Greek letter, λ. The failure rate is expressed as an average frequency of failure, i.e. a number of failures per unit of time.

The reliability of the DP system has been addressed in IMO MSC/Circ.645 since 1994 [1]. It states: A DP-system consists of components and systems acting together to achieve sufficiently *reliable* position keeping capability. The necessary ***reliability*** is determined by the consequence of a loss of position keeping capability. The larger the consequence, the more *reliable* the DP-system should be.

To achieve this reliability philosophy, IMO MSC/Circ.645 defines three DP equipment classes [1].

For *Equipment Class 1*, loss of position may occur in the event of a single fault.

For *Equipment Class 2*, loss of position is not to occur in the event of a single fault in any active component or system. Normally static components will not be considered to fail where adequate protection from damage is demonstrated.

For *Equipment Class 3*, loss of position should not occur given the following single failures, i) failures as specified in class 2, and static component failure is included, ii) failure of all components in any one watertight compartment from fire or flooding, iii) failure of all components in any one fire sub-division from fire or flooding.

The definition in IMO MSC/Circ.645 does not provide any expected failure rate for each DP equipment class. It was probably difficult to prescribe DP reliability requirements quantitatively in 1994. The DP vessel population was much less comparing to the number nowadays. According to Bierman [4], the number of DP vessels was 65 in 1980, 150 in 1985, and in 2011 the number was estimated to reach 2000 worldwide. The main challenge for the industry in 1994 was probably to ensure sufficient redundancy of the DP technical systems, so that station-keeping capability can be maintained after single failures. Quantification of failure rates for three DP equipment classes was not prioritized at that time. However, despite significant increase of the population of DP vessels, and significant DP technology development in the last 20 years, it remains uncertain or debatable in the industry regarding what are expected frequencies of position loss for DP vessels, in particular for DP class 2 and class 3 vessels.

For DP class 1 vessels the frequency of position loss is expected to be rather high due to sum of the likelihood of various single failures. It may range between several times per year to once every several years. However, for DP class 2 and class 3 vessels, is the position loss frequency low enough for DP class 2 vessels, and is it negligible if a vessel is designed to meet DP class 3 requirements? This paper is to provide answers and discussions around these questions.

Risk of position loss is intrinsic to all DP vessels. Given wide applications of DP vessels in complex marine operations, and sharp increase of DP vessel population, the DP risk in offshore marine operations is no longer negligible. There had been 26 collisions between offshore installations and visiting vessels on the Norwegian continental shelf during 2000-2010 [5]. Many of those vessels were DP vessels, and out of the 26, there were 6 incidents which had a very large consequence potential. In the UK offshore sector the ship/platform collision is also of concern [6]. The majority of the collision risk is from attendant vessels and not passing vessels. DP safety of offshore attendant vessels is clearly a risk element for attention. For DP mobile offshore drilling units, there is potential risk to people, environment and asset if emergency disconnection of Lower Marine Riser Package (LMRP) on top of Blowout Preventer (BOP) is not successful in case of a DP position loss. For diving support vessels and pipe-laying vessels, there are risks towards personnel (divers) and asset (pipes being laid) respectively in case of a position loss.

Modern risk management practices and safety regulations in the offshore industry are built on a quantitative framework. The Management Regulations by the PSA [7], §16 stipulates the requirements on suitable models, methods and data for conducting and updating the risk analyses. Such requirements shall have no exception when it comes to the DP risk involved in marine operations. This paper is to discuss how to quantitatively manage DP risk in offshore marine operations. The starting point is the failure rate of DP systems, i.e. the frequency of position loss. Case studies related to DP offloading and DP drilling operations are referenced to illustrate the method and values.

2 POSITION LOSS FREQUENCY

2.1 Position loss modes

The term *Position Loss* for DP vessels is defined as: the vessel loses, either temporarily or for an extended time, the ability to maintain its position or heading by means of thruster force, and consequently has a position or heading excursion which is beyond the allowable distance range.

The term position loss is operation-specific. The marine operations and allowable position excursion limits may vary significantly. For deep water drilling an excursion of 20 m of the rig may still considered as allowable. However, for heavy lifting and platform topside installation activity, an excursion of 5 m of the vessel may already result in catastrophic asset damage.

There are three basic failure modes of position loss, i.e. drive-off, drift-off and large excursion. *Drive-off* means a situation where active thruster forces driving the vessel away from its target position. The characteristic of drive-off is the abnormal thruster force. *Drift-off* means a situation where the vessel is incapable of maintaining its target position due to insufficient thruster forces/moment in relation to the environmental forces/moment. The characteristic of drift-off is the insufficient thrust in relation to the environmental loads.

Large excursion refers to a situation where the vessel is still under control of the DP system but has a temporary larger than average excursion of position or heading, because of large wind gust, wave, single thruster fault, worst case single failure, or degraded position information. Most of such events result in a rather limited vessel excursion, i.e. larger than the normal vessel footprint, but smaller than the allowable limit. However, in marine operations where the position excursion error margins are very small, e.g. 5-15 m, this failure mode becomes relevant or even critical.

A term *force-off* is often used in the industry. It refers to a situation where a change/increase in environmental conditions causes the vessel to operate outside its DP capability envelope and is forced off position due to insufficient thruster force. According to the definitions above in this paper, the force-off event can be categorized to either drift-off or large excursion.

2.2 Position loss data and frequency estimate

How frequent did position loss events occur on DP vessels in average? This is a challenging question to answer. Let us start with the position loss incident data. The most widely used data source is from the International Marine Contractors Association (IMCA). Every year the DP station-keeping incidents reported to IMCA are summarized and published. The latest issue of such report presents station-keeping incident data in 2013 [8]. The reporting of incident to IMCA is on a voluntary basis, and hence under reporting is inevitable. There are around 60 to 70 DP incidents reported yearly in average in last 10 years.

Vessel owners, DP equipment vendors, and oil companies may also have position loss data for DP vessels. However, such data are normally not open to the public, and can be scattered among various sources.

The IMCA station-keeping incident data are presented in simplified fault tree format to illustrate type of vessel, event sequences, primary and secondary causes, contributing factors, and event outcomes. The vessel name, owner or other sensitive data are protected and not disclosed. The data provides valuable experience sharing and lessons regarding causes to position loss. However, the IMCA incident reports do not contain any information regarding DP operational time nor DP position loss frequency estimates.

Incidents on DP mobile offshore drilling units (MODUs) from the reported IMCA station-keeping incidents during 2001-2010 are collected and analyzed by Chen & Nygård [9]. There were in total 57 position loss incidents on DP MODUs in this 10-year period, 5 drive-offs and 52 drift-offs.

It is observed that position loss incidents did occur on DP MODUs which are typically designed to meet with DP class 2 or class 3 requirements. Chen & Nygård [9] made efforts to further derive the frequency of position loss on DP MODUs. A frequency of 0.21 per DP year is found for position loss on DP MODUs (combined class 2 and class 3 DP vessels), i.e. 2.4E-05 per DP hour. It consists of 0.19 position loss per DP year for drift-offs and 0.03 position loss per DP year for drive-offs. The frequency is not negligible.

A number of assumptions were introduced when deriving this historical frequency, and hence the uncertainties are significant. The uncertainties are associated with the two key parameters, i.e. under-reporting of DP incidents, and the DP operational time relevant for the DP MODUs which reported

the incidents to IMCA. These two parameters are both rooted from the current DP incident reporting scheme in the industry.

2.3 Reliability of DP systems - Quantitative parameters

The reliability of DP systems, in terms of three DP equipment classes, is well recognized after the IMO MSC/Circ.645 came into force in 1994. It is the time for the offshore industry to consider quantitative parameters related to the reliability of DP systems. One of such parameters is the position loss frequency.

The need for transparent and accurate position loss frequency is clear, but has been continually neglected in the industry. Significant variations among different risk analysts regarding DP positon loss frequency also exist, which affects DP risk predictability. This hinders risk awareness and prevent effective risk management of DP operations, since the risk involved can be much underestimated. It is potentially harmful to the DP industry as well if the risk involved is much overestimated, since such may lead to choice of other station-keeping methods instead of DP.

Quality and consistent historical DP position loss frequency are lacking in the industry at present. Joint efforts among oil companies, marine contractors, industrial associations and risk analysts, are needed to improve the situation. Chen & Nygård [9] recommended an alternative DP incident data reporting scheme which combines incidents and DP operational time. This may provide a promising way forward. An illustration is shown in Figure 1. The * indicates additions to the current incident reporting regime.

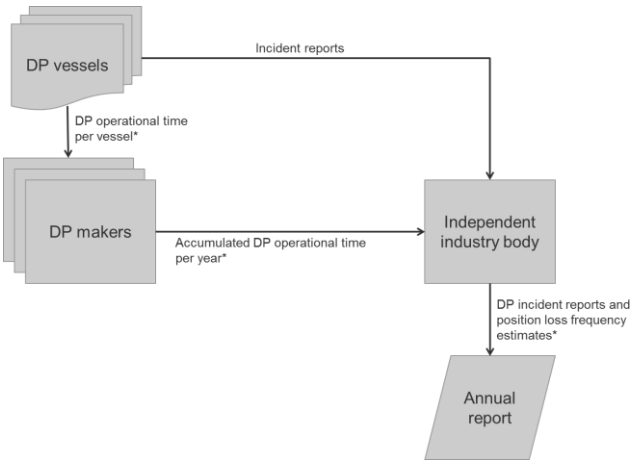

Figure 1. An alternative DP incident reporting scheme [9].

The proposed DP incident reporting scheme is summarized as follows:

1. Reporting of DP incidents should cover both position loss and redundancy loss events. This is largely the current practices by IMCA [10]. The focus is to encourage a good reporting culture.
2. Reporting of DP operational time can be achieved automatically based on the current technology and infrastructure from DP control system makers. There is no additional reporting work required by onboard personnel.
3. The position loss frequency can be derived by combining these two sources of data from a large number of DP vessels performing various types of marine operations.

The position loss frequency can be derived on a yearly basis for specific types of vessels, and be made accessible to the industry. This will fundamentally improve the understanding of the DP system reliability in the industry. It will also provide essential input for managing DP risk in offshore marine operations.

3 DP RISK IN MARINE OPERATIONS

3.1 Framework

The risk management process in ISO31000 [11] is illustrated in Figure 2. This framework is applied to the DP risk management in offshore marine operations.

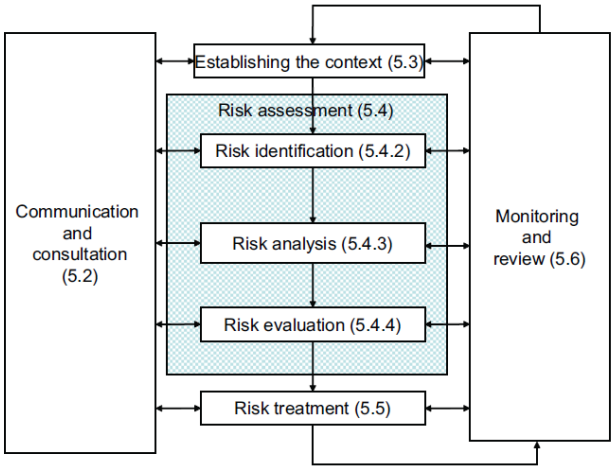

Figure 2. Risk management process [11]

The following three steps are essential for managing DP risk in marine operations:

1. *Hazard identification* - This step refers to identification of all possible position loss scenarios and their causes.
2. *Risk analysis and evaluation* - This step refers to analysis of position loss risk in terms of frequency and consequence for each scenario, and evaluation of risk against predefined acceptance criteria.
3. *Risk treatment* - This step refers to investigation of risk mitigation measures which eliminate and/or reduce the likelihood of position loss, as well as mitigate the consequence in marine operations given DP vessel position losses.

3.2 Qualitative approaches

Risk analyses and management activities of DP vessel in marine operations have traditionally been performed in a qualitative manner. The qualitative risk analysis and evaluation methods typically include, for example, Failure mode and effect analysis (FMEA) and consequence ranking (FMECA) of the entire DP system [12] & [13], Hazard Identification (HAZID) and Hazard & operability analysis (HAZOP) of operational procedures, activity specific operating guidelines (ASOG) [14], independent survey of DP system, various testing and FMEA proving trials, hardware-

in-the-loop testing of control software, training and certification of key DP personnel [15], and so on.

These qualitative approaches are effective to reduce failure of technical systems and increase reliability of human and operational barriers against DP incident. However, it is also fair to say that qualitative approaches are of "compliance verification" nature. They are targeted to verify DP system redundancy and ensure compliance with the designed DP equipment class, or to verify DP operators meet the prescriptive qualification requirements, or to ensure operations are performed and DP status are given according to the pre-defined criteria.

Being in compliance with all prescriptive requirements does not automatically guarantee the safety. There are novel design or novel operation which prescriptive requirements may not be suitable. Complacency in the industry, i.e. to disregard the position loss risk as long as a DP vessel is in compliance with the prescriptive DP class 2 or class 3 requirements, does exist. This calls for a quantitative approach where the risks caused by DP vessel position loss are analyzed and evaluated in terms of likelihood and consequences.

3.3 Quantitative approach

The quantitative approach for DP risk analysis and evaluation may consist of the following five main steps:

1. Assess frequency of position loss for the DP vessel involved in the marine operation.
2. Estimate probability of successful intervention by human or technical systems in case of position loss.
3. Establish vessel speed and distance profile involved in all possible position loss scenarios.
4. Calculate frequency of accident scenarios and estimate consequences, e.g. in terms of fatalities, impairment of main safety functions for the facility, or pollutions.
5. Establish the risk picture and associated uncertainties, evaluate against the risk acceptance criteria.

The approach has been described in [9]. An illustration of such quantitative DP risk analysis and evaluation process is shown in Figure 3. It is made in line with the practices of risk analysis and evaluation in the offshore industry, e.g. NORSOK, Z-013 [16]. Chapter 4 and 5 present two case studies in the past 10 years which are related to quantitative DP risk analyses. The values from such studies are demonstrated.

3.3.1 Frequency analysis

The frequency of accident is quantified based on the below frequency model for the DP risk in marine operations:

$$P(\text{accident}) = P(\text{position loss}) \times P(\text{failure of recovery})$$

The term P(accident) is the frequency of accident scenarios, e.g. collision, loss of well integrity, rupture of loading hose, damage of subsea installation, and so on.

The term P(position loss) is the frequency of DP vessel position loss. This is described in detail in Chapter 2. This quantitative parameter for the reliability of DP systems is the starting point for quantifying the accident frequency. The averaged historical frequency may be further adjusted to reflect vessel and operational specifics.

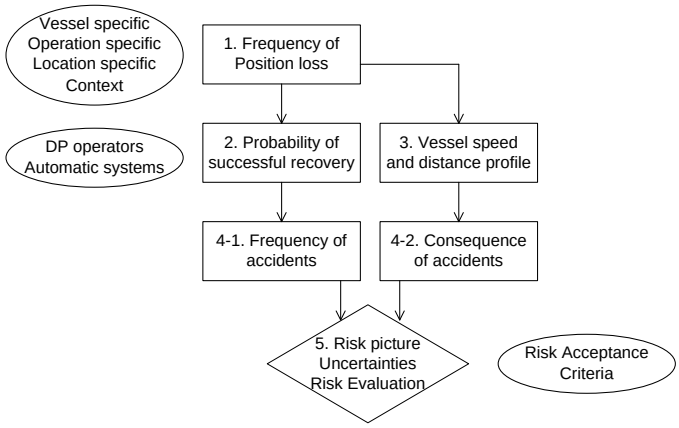

Figure 3. Quantitative DP risk analysis and evaluation process

The term P(failure of recovery) is the failure probability of human intervention, or failure of any automatic safety systems which are designed to mitigate or avoid accident given a position loss. The recovery actions performed by DP operators are mainly to arrest vessel movement in order to prevent collision, or to activate emergency disconnect sequence (EDS) to prevent damage of the well, and so on. The actions and timing for success are highly operation specific.

For some DP operations automatic safety systems have been developed to mitigate the consequence in case of human failure. For example, a Drive-off Prevention (DOP) function is developed to automatic manoeuver a DP flotel astern given detection of forward drive-off situation [17]. On some drilling units an automatic EDS system is developed to activate the emergency disconnect sequence if the DP control system detects that the rig position is outside the predefined red excursion limit.

By analyzing each position loss mode according to the above formula, accident scenarios in DP operations and their frequencies are quantified.

3.3.2 Consequence analysis

The consequence analysis involves analyses of vessel speed and distance profile involved in all possible position loss scenarios. This is normally done via simulated vessel movements which should be validated by full-scale trials if possible. Human intervention actions may affect vessel speed in a position loss scenario, and should also be addressed in the simulation model and trials. Thereafter, the consequence analysis is highly dependent on the nature of the marine operations. General guidelines on consequences related to the collision and well integrity accidents are further discussed below.

For collision accident, it involves analyses of the vessel impact speed and impact energy to the neighboring platform upon collision. It is often crucial to clarify other vital operating systems or areas which may be impaired by the collision, e.g. living quarter (LQ), flare tower, high pressure risers, or mooring lines and fairleads. Damage of such vital systems could escalate the collision consequence from minor

structural damage to catastrophic damage with total loss of an offshore installation.

For well integrity accidents upon DP position loss, the consequence evaluation may include the following, i.e. model the reliability of well barriers, assess their activations during the event chain after position loss, and quantify probabilities of failure-on-demand. Once blowout or well release frequencies are established, the corresponding well flow scenarios can be established. The risk of ignited blowout or well release, as well as the pollution risk caused by DP position loss, can then be assessed in the total risk analysis (TRA) of the installation.

4 DESIGN INHERENT SAFE DP OPERATIONS

A geostationary FPSO does not weathervane like a turret-moored ship-shaped FPSO. The offshore industry in the North Sea had limited experience for direct offloading from such geostationary installations before 2007. Since then, direct offloading operations from geostationary FPSOs by DP shuttle tankers have been developed, and key operational principles for such DP shuttle tanker offloading are presented in [18].

The shuttle tanker DP operations in the vicinity of a geostationary FPSO are designed with the inherent safety against position loss. An illustration is in Figure 4.

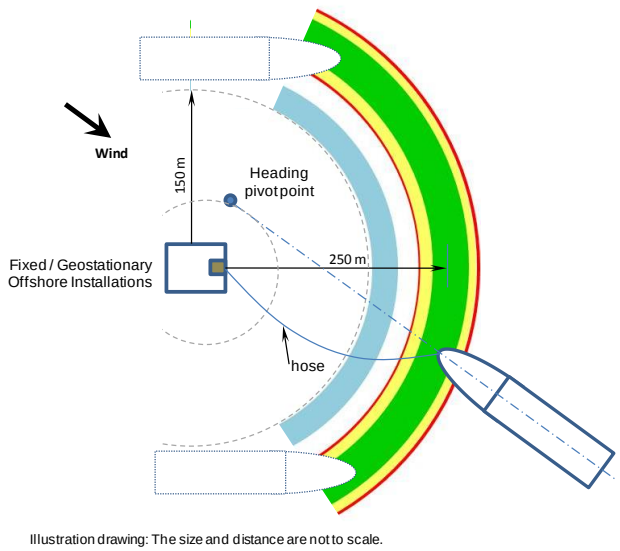

Figure 4. Principles of direct offloading [19].

The following two safety features are observed in Figure 4:

1. Shuttle tanker weather vanes with a heading pivot point which is away from the FPSO. In case of drive-off forward and no human intervention, the shuttle tanker will not hit the FPSO. This significantly reduces the collision risk exposure time during offloading operation.
2. Separation distance between shuttle tanker and FPSO is kept in a range of 200 m to 250 m. This provides adequate time window (close to 3 minutes) for DP operators to react in case a drive-off happens. According to the human reliability knowledge of DP operators [20], the success ratio of DP operator to intervene drive-off is high given such available time.

Regarding collision consequence, special FPSO hull design is developed to protect mooring fairleads. In case of shuttle tanker collision, either direct hit or in a glancing blow type of scenario, the tanker hull will not be possible to impact the mooring system.

Quantitative DP risk analyses had been performed during 2007 to 2014, with several revisions to support and verify the design of this novel offloading operations. Collision risk [19] and hose rupture and oil spill risk [21] are both addressed. It concluded that the direct offloading operation has a total collision frequency (6.43E-05 per year) that equals only 0.4% of the frequency for the equivalent tandem offloading (1.62E-02 per year).

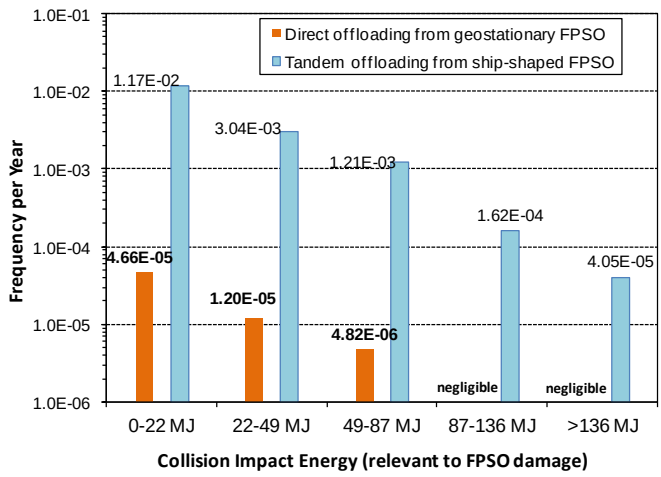

Figure 5. Collision risk picture comparison: direct offloading vs. tandem offloading [19].

In summary, the value of analyzing DP risk quantitatively is demonstrated during the design of a novel DP operation. The general principles of direct offloading have been applied to, e.g. Goliat FPSO and Aasta Hansteen spar. The concept of direct offloading brings inherent safety for DP shuttle tankers operating in the vicinity of an offshore production installations.

5 OPTIMIZE DP OPERATIONAL PHILOSOPHIES

The CAT-D drilling rigs are dynamically positioned semisubmersible drilling units designed for operations on the Norwegian Continental Shelf. The rigs are delivered in 2015 and 2016, and are designed to meet DYNPOS-AUTRO and DYNPOS-ER notations. There are three DP redundancy groups, and single failures including fire or flooding will in maximum result in loss of one DP redundancy group, i.e. two diesel generators (DG), or two connected 11kV switchboard sections, and/or two thrusters. Within each DP redundancy group the rig is designed with extra fault tolerance capability against single failures. Except for fire/flooding events and failures related to 11kV bus coupler, single failures can only result in loss of one DG, or one 11 kV switchboard section, and/or one thruster. This represents a significant step forward

to reduce the vulnerability of DP class 3 system towards single failures.

The enhanced fault tolerance in the DP system design, and in particular, the most likely expected single failure effect, i.e. loss of one thruster in station keeping, is the motivation for a quantitative DP risk analysis. The core question to answer is: Can CAT-D rigs be operated safely based on single failure effect of losing one thruster? Or, after significant extra investment on the design and construction of the rig, should rig operator still plan the DP operation in a similar way as other contemporary DP class 3 rigs, i.e. based on single failure effect of losing two thrusters? The quantitative risk analysis during 2013-2015 provided answers to above questions. Key results are published at 2015 MTS DP conference [22].

The objective of the quantitative DP risk analysis is to evaluate the risk of loss of well integrity in the Troll field due to DP position loss. The focus is to analyze and compare risks given two different DP operational philosophies for the CAT-D rig, i.e. WSF2 (which means DP operations are based on the worst case single failure of losing two thrusters), vs. WSF1 (which means DP operations are based on the worst case single failure of losing one thruster).

The historical fire and flooding frequency is derived for mobile offshore drilling units. Fire/flooding as well as 11kV bus coupler failures can cause loss of two thrusters, and rig will slowly drift off given DP operation based on WSF1. The frequency of such slow drift-off on the CAT-D rig is found to be 2.6E-04 per year for operations in the Troll field.

Such slow drift-off scenarios are simulated by time-domain simulations using rig specific model and environmental conditions in the Troll field. Event tree analyses of rig position loss and barriers, i.e. automatic EDS, human operator, and independent ADS system, are performed.

The frequency of impairment of well integrity given such slow drift-off is calculated to be in the range of 1.3E-07 per year to 2.3E-07 per year. It indicates that the increase of risk for impairment of well integrity given DP operation based on WSF1 instead of WSF2 is negligible. The conclusion is that DP operations based on WSF1 have equivalent safety level for well integrity as DP operations based on WSF2 in the Troll field. Subsequently, WSF1 is implemented as a valid DP operational philosophy in daily DP operations on all CAT-D rigs.

In summary, the value of analyzing DP risk quantitatively is demonstrated by determining an optimum DP operational philosophy for an advanced DP system design.

6 CONCLUSIONS AND RECOMMENDATIONS

The reliability of DP systems, in terms of three DP equipment classes, is defined in the IMO MSC/Circ.645 since 1994. It is the time for the offshore industry to consider quantitative parameters related to the reliability of DP systems. One of such parameters is the position loss frequency.

Despite significant increase of the population of DP vessels, and significant DP technology development in the last 20 years, it remains uncertain or debatable in the industry regarding what are expected frequencies of position loss for DP vessels, in particular for DP class 2 and class 3 vessels. The only thing for certain is that position loss incidents did occur on DP class 2 and class 3 vessels in the past. Even with modern DP technology and well-trained operators, such position loss will most likely continue to happen.

Joint efforts among oil companies, marine contractors, industrial associations and risk analysts, are needed to improve the situation. An alternative DP incident reporting scheme is proposed for transparent and consistent position loss frequency data. This will fundamentally improve the understanding of the DP system reliability in the industry. It will also provide essential input for managing DP risk in offshore marine operations.

A frame work for managing DP risk in offshore marine operations is described in the paper. Qualitative approaches are discussed and deficiencies are outlined. Complacency does exist in the industry which is to disregard the position loss risk as long as a DP vessel is in compliance with the prescriptive DP class 2 or class 3 requirements. A 5-step quantitative approach is recommended in the paper where the risks caused by DP vessel position loss are analyzed and evaluated in terms of likelihood and consequences.

Two case studies, i.e. design inherent safe DP offloading operations, and optimize DP operational philosophies in DP drilling operations, are presented to illustrate the values of analyzing DP risk quantitatively.

ABBREVIATIONS

ASOG	Activity Specific Operating Guideline
BOP	Blowout Preventer
DG	Diesel Generator
DP	Dynamic Positioning
DOP	Drive-off Preventer
EDS	Emergency Disconnect Sequence
FMEA	Failure Mode and Effect Analysis
FMECA	Failure Mode, Effect and Criticality Analysis
FSOG	Field Specific Operating Guidelines
HAZID	Hazard Identification
HAZOP	Hazard and Operability Analysis
IMCA	International Marine Contractors Association
IMO	International Maritime Organization
LMRP	Lower Marine Riser Package
LQ	Living Quarter
LSOG	Location Specific Operating Guideline
MODU	Mobile Offshore Drilling Unit
PSA	Petroleum Safety Authority (of Norway)
TRA	Total Risk Analysis
HSE	Health and Safety Executive
WSF1	Worst case Single Failure of losing 1 thruster
WSF2	Worst case Single Failure of losing 2 thruster
WSOG	Well Specific Operating Guideline

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Chapter 16

Session 14: Floating Arctic Structures

16.1 Presentation by Richard McKenna

NTNU Norwegian University of Science and Technology

International Association of Oil & Gas Producers

Sea ice management and reliability of floating structures

Richard McKenna, R.F. McKenna Associates
Brian Wright, B. Wright Associates

The 3rd Offshore Structural Reliability Conference
OSRC2016
14-16 September, Stavanger, Norway

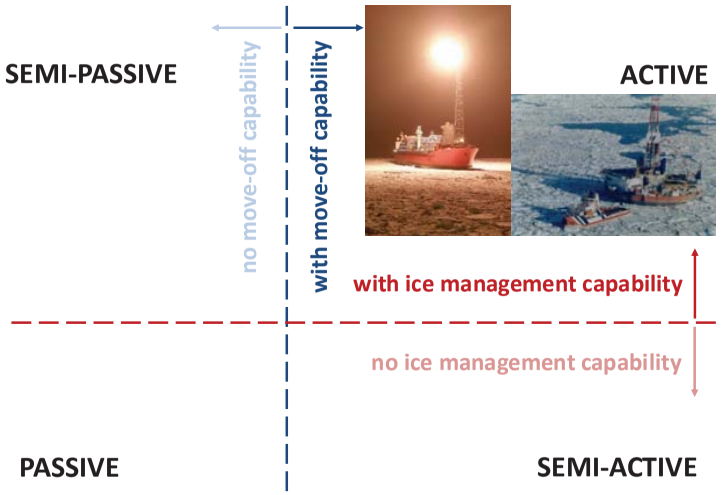
Context

- ISO/DIS 19906 (first revision) scheduled for issue in March 2017
- acknowledge contributions of SC7/WG8(arctic structures)/TP7(floaters) membership
- acknowledge contributions of colleagues involved with sea ice and iceberg management systems

Scope of presentation

- ISO 19906 strategy
  - allows operational measures to be used to reduce ice actions
  - hull as a safe haven principle (safety of life, hydrocarbons)
  - requirements to ensure that ALS are satisfied
- effectiveness of operational measures for reducing sea ice actions
  - detection, threat assessment, and drift forecasting
  - ice management
  - alert system

Floating structure categorization in ISO 19906



Strategy for floaters in ISO 19906

- ensure an equivalent level of safety to that for fixed structures (in a general sense)
- can apply different basic strategies
  - fixed structure approach, but with altered ice environment and associated uncertainties
  - demonstrate ability to mitigate risk through shut-in and move-off in the context of an alert system
- focus on active systems because ice management alone is insufficient for ensuring that EL actions are resisted by the stationkeeping system in many ice environments

5

Operational procedures in ISO 19906

- operational procedures need to be conducted under an ice alert system
  - mandatory responses to specified ice situations
  - response times set to ensure safety of personnel and security of hydrocarbons
  - ULS and ALS criteria are met by the operational envelope of ice situations
- success needs to be verified under actual conditions
  - sea ice floe size reduction
  - iceberg deflection
  - learning by doing (see ISO 35104, ice management)
    - set restrictive operational constraints initially
    - relax as experience is gained and documented
    - intended success achieved over design service life

6

Requirements for safety of floaters

- ALS criteria
  - frequency of ice events (features and situations) leading to failure (loss of life, major spills) is less than 10^{-4} per year
  - probability of failure to assess potential ice events, shut-in (+ move-off for active and semi-active sustems) is less than 10^{-4} per year
- ULS criteria
  - EL (extreme-level) actions* are less than the capacity of the station-keeping system: approach is the same as for fixed structures
  - EL actions* are limited by the capacity of stationkeeping system: approach outlined in ISO 19901-7 (station-keeping systems) and ISO 19904-1 (floating structures)

  * operational measures may be used to alter the ice environment experienced by the structure, resulting in reduced ice actions

7

Ice hazards

- detection system needs to be able to see and characterize hazardous ice features and situations (to a distance or travel time consistent with alert system response times)
- threats need to be assessed relative to alert system requirements
- drift forecast accuracy needs to be demonstrated and quantified
- pack ice pressure situations and rapid changes in drift direction need to be forecast, monitored, and dealt with by ice management fleet

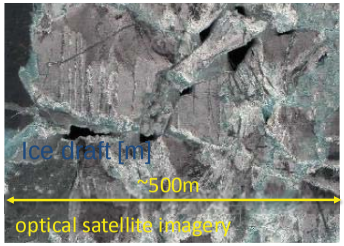
8

Downtime considerations

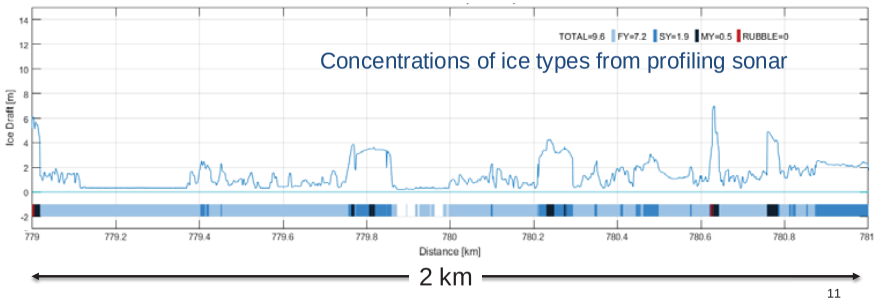
- want to limit downtime (for drilling or production) to realistic limits
  - frequent move-offs are not feasible
  - should be able to comfortably drill a well in a single season
  - recognize the potential difficulty of getting back on station
- strategy in ISO 19906
  - ensure basic safety
  - up to user to design a system with acceptable downtime

9

Ice characterization

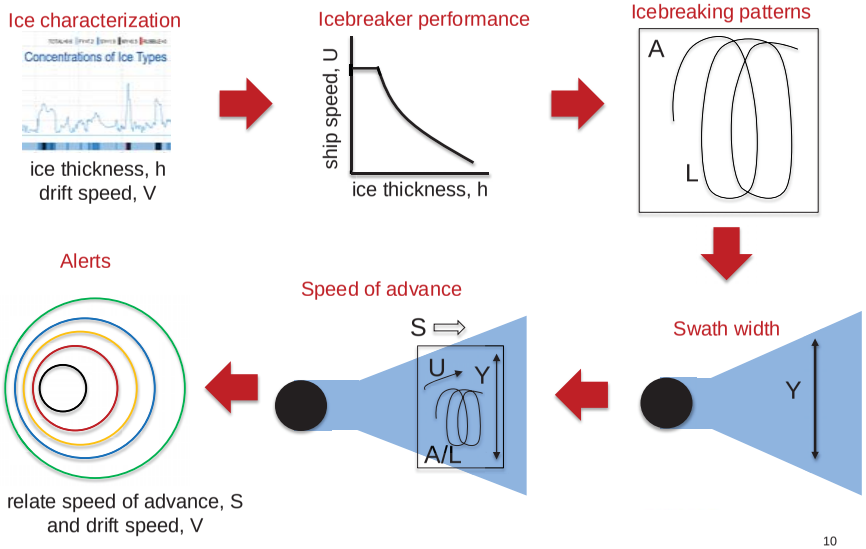


- imagery (hindcasts or real time)
- ice profiling sonar (hindcasts)
- thickness, h
- drift, V, including reversals
- floe size, D
- ice concentration, C
- other physical environmental factors



11

Sea ice management characterization



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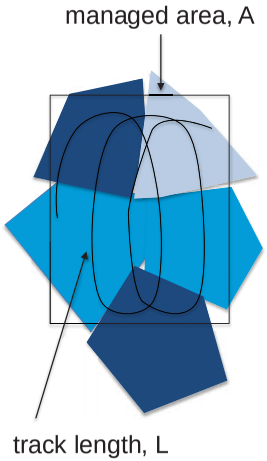
Icebreaker performance



different icebreaking speeds in different ice conditions (some conditions may require ramming)

U = f(h, D, C, vessel performance characteristics)

average speed, U

average speed weighted by the proportions of different ice conditions within the managed area

12

Icebreaker performance

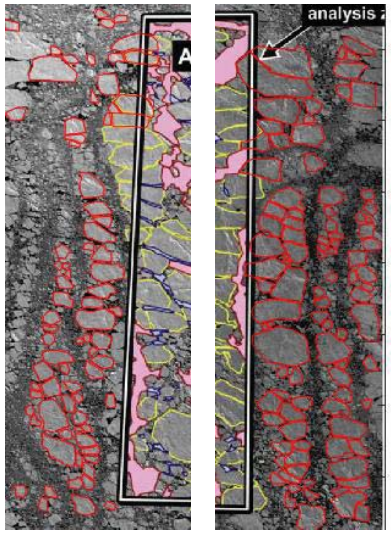
- forward advance through different ice types and thicknesses
- turning, backing and ramming

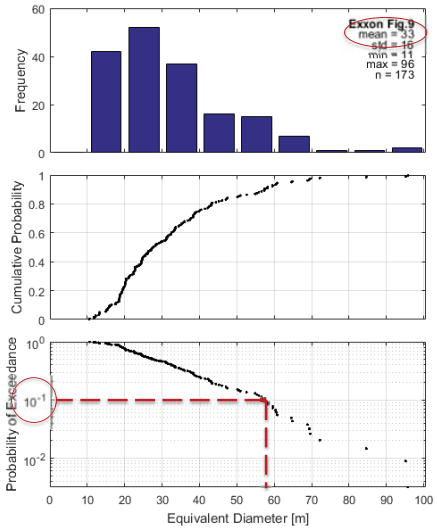
KEY ASPECT IS TIME TAKEN TO BREAK ICE OVER A SPECIFIED TRACK LENGTH

- other factors
  - ice ridges and rubble
  - visibility
  - pack ice pressure

13

Icebreaking patterns: ExxonMobil, Fram Strait





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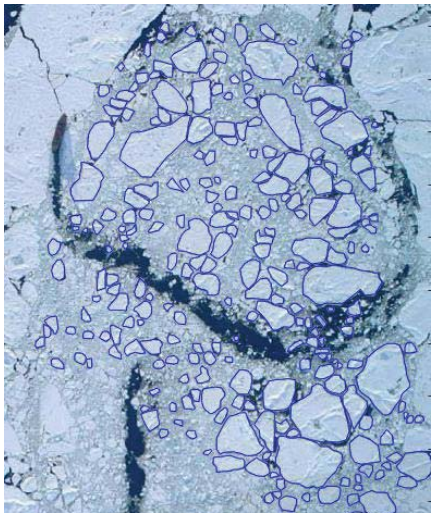
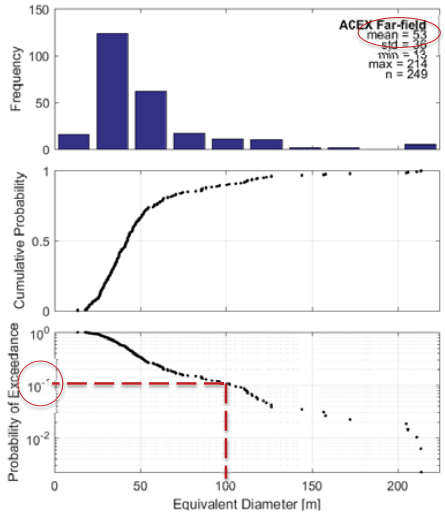
Icebreaking patterns: ACEX coring experiment





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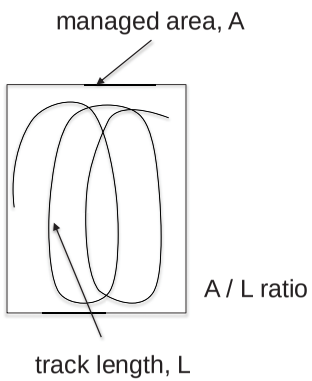
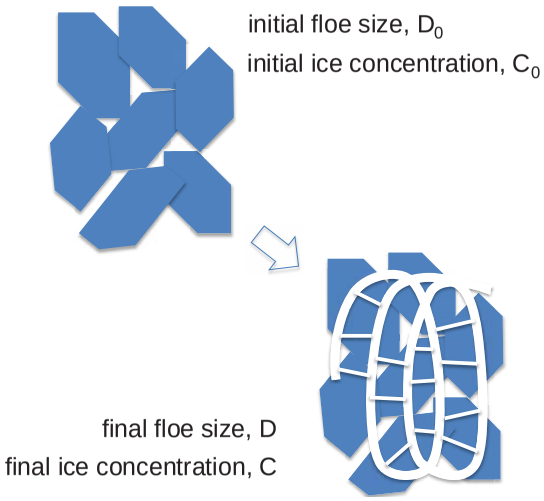
Icebreaking patterns





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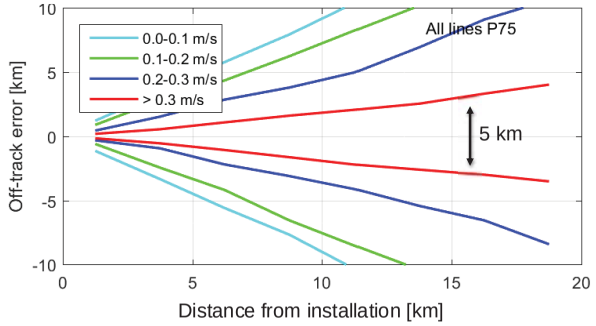
Swath width

- the width of ice that needs to be managed updrift of the installation depends on the uncertainty in the direction of drift
- the off-track errors below are based on forecasts for ice at different drift speeds and distances from the installation



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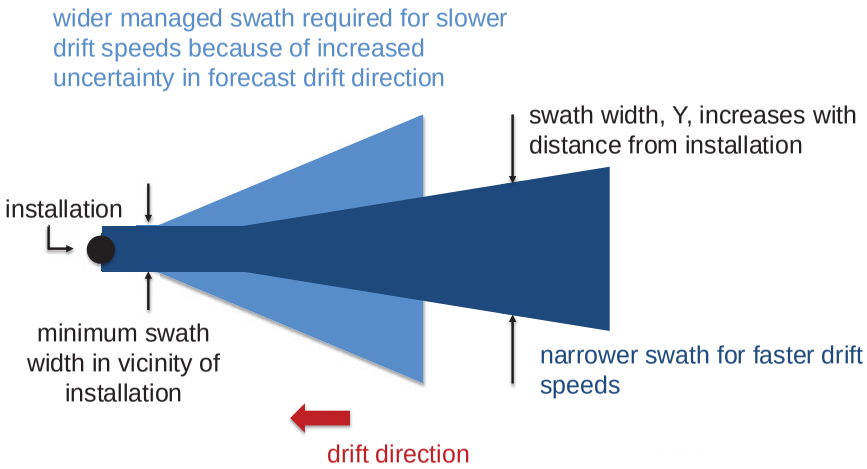
Swath width



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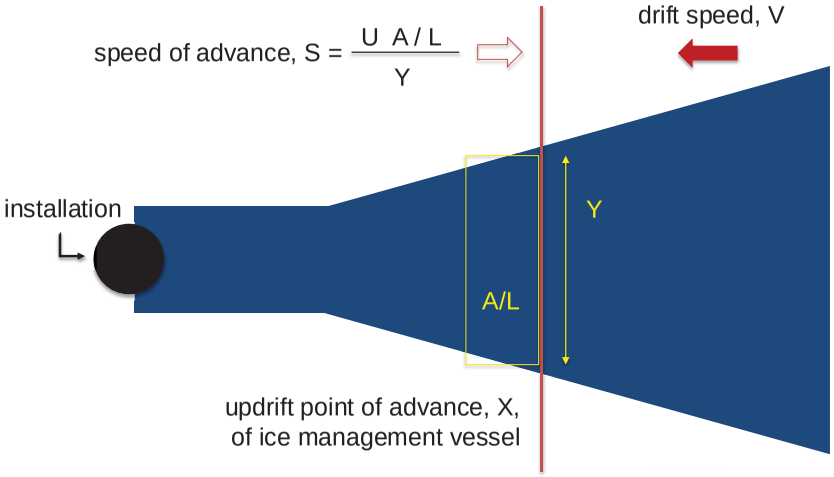
Ice management speed of advance



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Alert status (drilling example)

alert time is the time available to continue well operations

AT = HT – (ST + MT)

where:

HT = time for ice hazard to reach installation

ST = time to secure well (e.g. 3h)

MT = move-off time (e.g. 1h)

black alert, HT = e.g. 1hr
red alert, AT = 0hr
yellow alert, AT = e.g. 6hr
blue alert, AT = e.g. 12hr
no alert

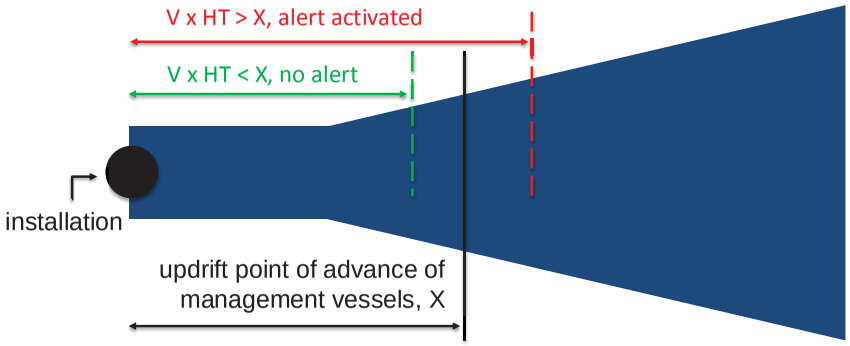
20

Alert status

an ice alert is activated when:

V x HT > X (point of advance of ice management vessels)

i.e. when the hazard time, HT, multiplied by the ice drift speed, V, is greater than the distance to which the ice can be managed



Challenges

- pack ice pressure situations
- rapid changes in drift direction
- undetected ice features or situations

→ avoid through on-site experience, contingency, special ice management techniques, and move-off capability

→ there are no substitutes for experience and preparedness; modelling and simulation can only take one so far

Way forward

- characterization of uncertainties in success of ice management operations through on-site experience
- continue to challenge ourselves to improve on the strategy and requirements in our standard(s)
  - effective to ensure safety
  - simple to ensure proper use and understanding
  - specificity to ensure ease of use
  - generality so that appropriate solutions are found

16.2 Presentation & article by Kaj Riska & Robert Bridges

TOTAL
COMMITTED TO BETTER ENERGY

LIMIT STATE DESIGN IN ICE CLASS RULES FOR SHIPS AND STANDARDS FOR ARCTIC OFFSHORE STRUCTURES

Kaj Riska & Robert Bridges Total SA E&P/DSO/TEC/GEO

The 3rd Offshore Structural Reliability Conference
OSRC2016
14-16 September, Stavanger, Norway

TARGETS OF PRESENTATION:

- Analyse the limit state definitions in ice rules for ships and offshore structures
- Illustrate the definitions and their use for design temperature and steel shell structure design for ice.

Expected outcomes:

→ Understanding of the implicit nature of the limit state definitions

→ Show the differences in various limit states used.

LIMIT STATE DEFINITION:

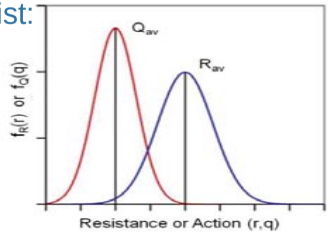
LRFD - Load and Resistance Factor Design

$\Phi R_n \geq \Sigma_k \gamma_k Q_{kn}$; structural resistance larger than load actions

The nominal resistance and action need to be defined at some pre-defined probability; usually given by return period.

Several approaches to return period exist:

- Serviceability Limit State (SLS) – normal actions, often once in 10 years
- Ultimate Limit State (ULS) – extreme actions once in 100 years
- Accidental Limit State (ALS) – abnormal actions once in 10^4 years



For ship wave loading the approach often is to state the number of waves encountered in ship life time; this is about 10^8.

RULES AND STANDARDS ANALYSED:

- Common Structural Rules (CSR)
- Finnish-Swedish Ice Class Rules (FSICR)
- IACS Polar Class rules (PC)
- Arctic Offshore Structures (ISO 19906)

The analysis concentrates on definition of the ice action, limit state (capacity) definition and return period definition.

STANDARDS FOR OFFSHORE STRUCTURES

Ice actions (ISO 19906)

Each of the following conditions shall be considered, and the governing ones shall be used to determine ice actions:

a. quasi-static actions due to level ice (first-year, rafted or multi-year), where inertial action effects within the structure can be neglected;
b. dynamic actions due to level ice (first-year, rafted or multi-year), where inertial action effects within the structure are influential and a dynamic analysis is required;
c. quasi-static actions due to ice rubble and ridges, where inertial action effects within the structure can be neglected;
d. impacts from discrete features such as icebergs, ice islands and large multi-year or first-year ice features;
e. quasi-static actions from features lodged against the structure, driven by the surrounding ice or directly by metocean actions;
f. adfreeze action effects, including the frozen-in condition; and
g. thermal action effects.

STANDARDS FOR OFFSHORE STRUCTURES

Limit state definition

Serviceability limit states
Exceedance of SLS results in the loss of capability of a structure to perform adequately under normal use. The specification of actions for SLS is generally the owner's responsibility, except for considerations that can lead to long-term structural degradation, such as corrosion of reinforcement in concrete.

Ultimate limit states
The ULS requirement ensures that no significant structural damage occurs for actions with an acceptably low probability of being exceeded during the design service life of the structure.

Abnormal (accidental) limit states
The ALS requirement is intended to ensure that the structure and foundation have sufficient reserve strength, displacement or energy dissipation capacity to sustain large actions and other action effects in the inelastic region without complete loss of integrity. Some structural damage can be allowed for ALS.

STANDARDS FOR OFFSHORE STRUCTURES

Return periods

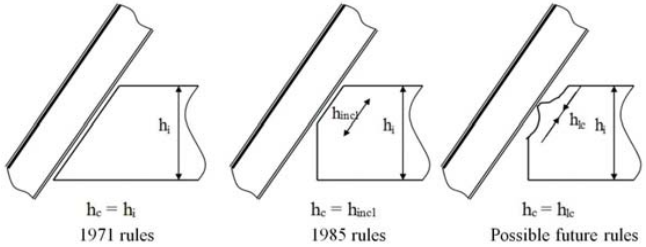
Serviceability-level ice events
Unless the owner has specified otherwise, the characteristic value of the SLIE used for SLS shall be determined based on an annual probability of exceedance not greater than 10^{-1}.

Extreme-level ice events
The representative value for actions arising from extreme-level ice events (ELIE) shall be determined based on an annual probability of exceedance not greater than 10^{-2}.

Abnormal-level ice events
The representative value for actions arising from the ALIE shall be determined based on an annual probability of exceedance related to the exposure level. For L1 structures, this shall be determined based on an annual probability of exceedance not greater than 10^{-4}.

STANDARDS FOR SHIPS

FSICR: Load height with constant value of the line load q at about q = 1 MN/m



PC: Scenario based design

SHIP/ICE INTERACTION SCENARIO BY REGION/SEASON
ARCTIC - EASTERN HEMISPHERE
RUSSIAN

SHIP/ICE SCENARIO - WINTER		REGIONS					
		Barents	Chukchi/ Bea	East Siberian	Kara	Laptev	Northern Barents	Southern Barents
Level ice	1. Proceeding continuously ahead in level ice	3	3	3	2	2	2	3
	2. Going astern or maneuvring in level ice	3	3	3	2	2	2	3
Ramming	1. Ramming in level ice	2	2	4	4	4	2	3
	2. Ramming ridges	4	4	4	4	4	4	3
	3. Ramming and manoeuvring in ridges and hummock field	5	5	5	5	5	4	3
Open Pack	1. Glancing impact on a floe	0	0	0	0	0	0	4
	2. Reflected impact from one floe to another	0	0	0	0	0	0	4
	3. Striking a floe on the quarter during a turn	0	0	0	0	0	0	4
Wedging	1. Wedging between two floes (A) or a narrow lead (B)	2	2	4	4	4	3	2
	2. Ship enters a narrow lead, rides up on one side, falls sideways	2	2	4	4	4	3	2
Dangerous	1. Striking a concealed multi-year floe or growler	3	3	3	2	3	4	2
	2. Striking the underwater projection of an iceberg	0	0	0	2	1	3	3
	3. Proceeding in open pack at speed-making contact with heavy ice	0	0	0	2	1	3	3
	4. Ramming a growler among whitecap waves	0	0	0	0	0	0	2
	5. Ramming (A) shoulder impact (B) iceberg, floeberg or ice island	0	0	0	2	1	3	3
Icebreaker	1. Striking floes in the track of an icebreaker	5	5	5	5	5	5	2
	2. Heavy floes pushed against an escorted ship by an icebreaker	5	5	5	5	5	5	2
	3. Escorted ship following an icebreaker unable to negotiate turns in track	5	5	5	5	5	5	1

0; nil
5; very high

STANDARDS FOR SHIPS

CSR: typical ASD

Acceptance criteria set	Plate and local support members	Primary support members	Hull girder members
AC1	70-80% of yield stress	70-75% of yield stress	75% of yield stress
AC2	90-100% of yield stress	85% of yield stress	90-100% of yield stress
AC3	Plastic criteria	Plastic criteria	NA

FSICR: Implicit ASD (based on stress)

Based on ice damage feedback: If the full scale measurements are analysed, it can be inferred that the WSD is used with the elastic limit as limiting stress.

PC: Implicit ASD (based on strength)

Local [plating and] frames…are to be dimensioned such that the combined effects of shear and bending do not exceed the plastic strength of the member

STANDARDS FOR SHIPS

CSR:

Probability level 10^{-8} (in e.g. 5.4.2.4 of CSR for oil tankers)

FSICR:

The return period for ice action is less clear but for frames it seems to be 1 year and for plating even less

PC:

In the technical background material for the IACS rules there is mention that the ice actions can be considered to be based on an annual probability level of 1.0 (return period one year)

EXAMPLE 1: DESIGN TEMPERATURE, T_D

ISO 19906

The design air temperature is specified by: *The lowest anticipated service temperature (LAST) shall be defined in accordance with 3.48*. The referenced paragraph states: *LAST: Minimum hourly average extreme-level (EL) air temperature*. This point is further emphasised in 11.9.1: *...the LAST value used for material selection and testing should be defined in accordance with this International Standard...* and also in topsides chapter 15.1.1.2: *Structural materials should be selected based on the requirements of 11.9 and the ... LAST.* Thus the design temperature is the lowest temperature based on a return period of 100 years.

EXAMPLE 1: DESIGN TEMPERATURE, T_D

IACS UR S6

The design temperature t_D is to be taken as the lowest mean daily average air temperature in the area of operation'. The daily low temperature is taken for each day of the year over a minimum 10 year period.

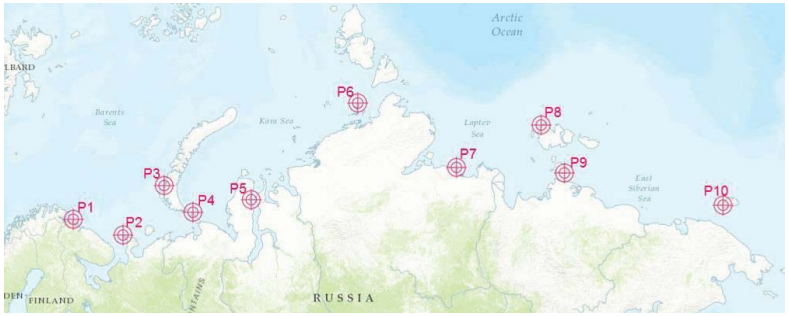
'For the purpose of issuing a Polar Ship Certificate in accordance with the Polar Code, the design temperature t_D shall be no more than 13°C higher than the Polar Service Temperature (PST) of the ship.'

IMO Polar Code

Polar Service Temperature (PST) means a temperature ... which shall be set at least 10°C below the lowest Mean Daily Low Temperature (MLDT) for the intended area and season of operation in polar waters. Systems must be 'fully functional' in or materials 'suitable for operation' at this temperature.

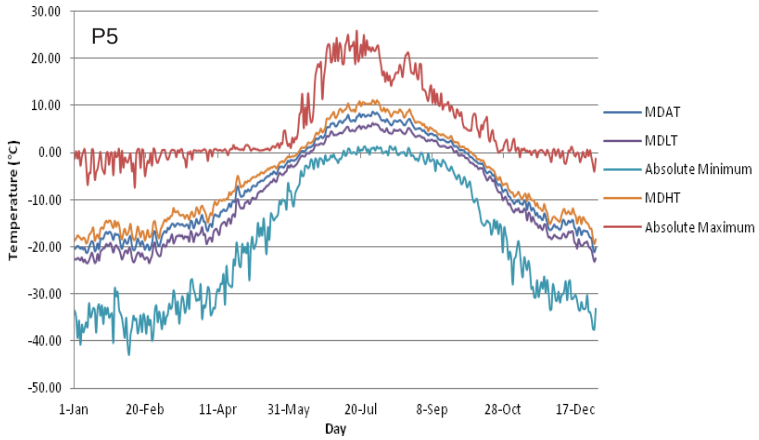
EXAMPLE 1: DESIGN TEMPERATURE, T_D

Application: Russian Arctic



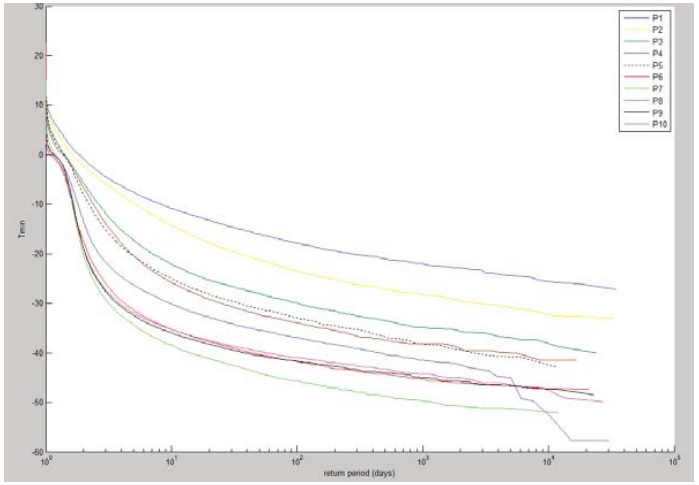
EXAMPLE 1: DESIGN TEMPERATURE, T_D

Application: Russian Arctic



EXAMPLE 1: DESIGN TEMPERATURE, T_D

Application: Russian Arctic



EXAMPLE 1: DESIGN TEMPERATURE, T_D

Application: Russian Arctic

Design Temperature (P5)		°C	Date
Lowest MDLT		-23.5	10 January
Polar Service Temperature		-33.5	
IACS UR S6		-34.3	
Absolute Minimum		-42.8	08/02/1979
Return Period (years)	*100*	-43.4	
	10 000	-52.8	

EXAMPLE 2: LOCAL STEEL STRUCTURES

ISO 19906

Distinction between 'global' and 'local' ice pressure. The global ice pressure is given as:

$$p_G = C_R \left(\frac{h}{h_1}\right)^n \left(\frac{w}{h}\right)^m$$

where w is structure width, h ice thickness and constants h_1 = 1m, n=-0.5+h/5 (h<1m) and m=-0.16. The constant C_R accounts for ice strength and also statistics. Its value is stated to be 2.8 with a return period of 100 years (ULS case) and 3.36 with a return period of 10,000 years (ALS case).

EXAMPLE 2: LOCAL STEEL STRUCTURES

ISO 19906

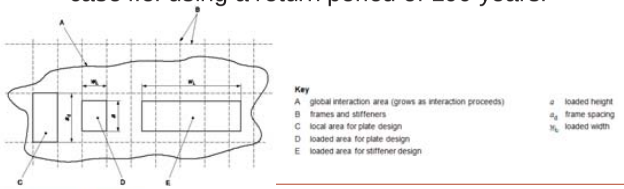
The design ice load patch for local transverse frames is assumed to be 0.4h in height with width w_L = s for plate and 4s for transverse frames, where s is the frame spacing. Then the force on the design area is given as (in MN):

$$F_L = 3.72 \cdot \sqrt{0.4h} \cdot w_L = 2.35 \cdot \sqrt{h} \cdot w_L$$

and the pressure (in MPa, h>0.35m):

$$p_F = 5.875\, h^{-0.5}$$

The ice thickness to be used corresponds to ULS case i.e. using a return period of 100 years.



EXAMPLE 2: LOCAL STEEL STRUCTURES

ISO 19906

SLS: 'subjected to localised ice damage', '[without] the loss of capability to perform adequately under normal use';

ULS: 'no significant structural damage', 'desig procedures shall be based primarily on linear elastic methods', 'both local and lobal actions shall be considered';

ALS: 'structural components are allowed to behave plastically'.

EXAMPLE 2: LOCAL STEEL STRUCTURES

FSICR

Quantity	Input parameters
Nominal ice pressure p_0	
↓	
Ice pressure p ←	Power P and displacement Δ Hull area and ice class Load length l_a / structural member
↓	
Load height h ←	Ice class
↓	
Line load $q = p \cdot h$	
↓	
t plating Z, A frames, stringers F web frames	

EXAMPLE 2: LOCAL STEEL STRUCTURES

FSICR

The FSICR use a factor to define the ice pressure as a coefficient dependent on the load length, c_a. Each structural member has an associated load length l_a – this is the length of the load that influences the response (stress) in the member. The load length coefficient is defined as:

$$c_a = \sqrt{\frac{l_0}{l_a}} \qquad \text{max 1.0 and min 0.35}$$

where the reference length is l_0 = 0.6 m.
In principle, each structural member should be designed by trying all load lengths and then selecting the design case to be the length that gives the maximum stress. The load lengths are given in the rules as

EXAMPLE 2: LOCAL STEEL STRUCTURES

FSICR

Structural member	Type of framing	Design load length l_a [m]
Shell plating	Transverse	Frame spacing
	Longitudinal	1.7 · frame spacing
Frames	Transverse	Frame spacing
	Longitudinal	Span of frame
Ice stringer		Span of stringer
Web frame		2 · web frame spacing

The total ice load for each structural member is then taken as the ice pressure times a load length l_a that depends on the width (horizontal span or spacing) of each structural member and load height that is a class factor.).

EXAMPLE 2: LOCAL STEEL STRUCTURES

IACS PC

The formula included in the IACS Polar Ship Rules for bow ice force uses an assumed value for ship speed and ice thickness (which increase with ice class) given as follows:

$$F_i = C_f D^{0.64}$$

where C_f is a factor accounting for hull shape and ice class (for the two lowest classes it is approximately 1.12 and 1.05) and D ship displacement. Bow ice pressure is given as:

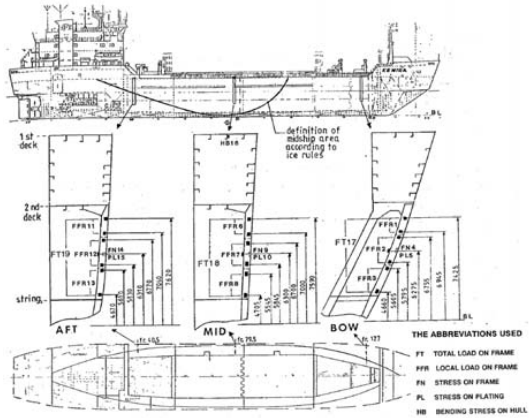
$$P_i = C_p F_i^{0.22}$$

where again the factor C_p depends on hull shape and ice class (for the two lowest classes it is approximately 1.74 and 1.65).

EXAMPLE 2: LOCAL STEEL STRUCTURES

Application: Baltic – using ice classes IA Super and PC6

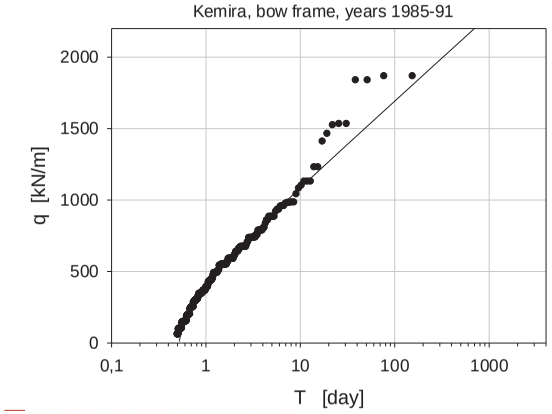
Data used: MT Kemira, measurements winters 1985 - 92



EXAMPLE 2: LOCAL STEEL STRUCTURES

Application: Baltic – Ice classes IA Super and PC6

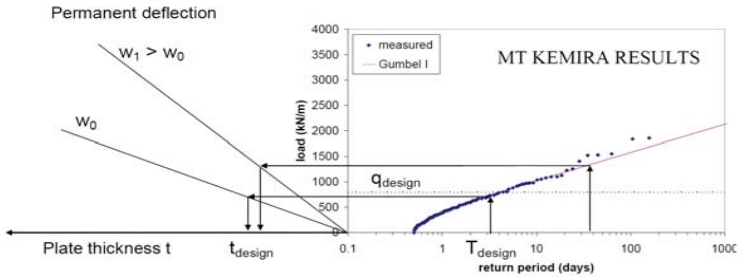
Data used: MT Kemira, measurements winters 1985 – 92;
32 ice days per year



EXAMPLE 2: LOCAL STEEL STRUCTURES

Application: Baltic – Ice classes IA Super and PC6

Limit state definition and analysis based on the data – using line load q



Statistical analysis gives ice action versus return period T; q = g(T)
Structural analysis gives a relationship between ice action, response denoted w and scantlings d;
These can be combined to give scantlings versus limit response and return period.

EXAMPLE 2: LOCAL STEEL STRUCTURES

Application: Baltic – Ice classes IA Super and PC6

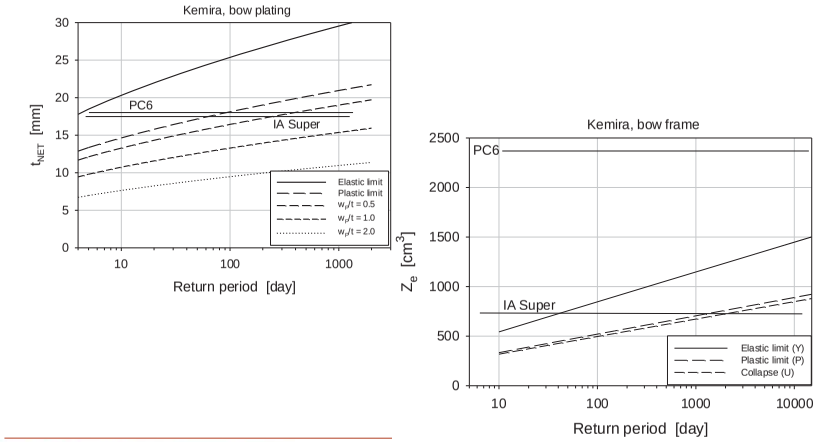
Limit state definition and analysis based on the data – using line load q

Limit state (label)	Plating	Frames
Elastic (Y)	Stress reaching the yield stress σ_y somewhere in the plate	Stress reaching the yield stress σ_y somewhere
Plastic (P)	Stress distribution reaching full plasticity somewhere in the plate; permanent deformation still zero	2-hinge formation at the frame supports
Ultimate (U)	Permanent deformation (w_P) reaching a specified value	3-hinge formation at the frame supports and the mid-span

EXAMPLE 2: LOCAL STEEL STRUCTURES

Application: Baltic – Ice classes IA Super and PC6

Limit state definition and analysis based on the data – using line load q



EXAMPLE 2: LOCAL STEEL STRUCTURES

Application: Baltic – Ice classes IA Super and PC6

Rule values (T_h = 100 years gives about h = 1 m) and corresponding Return periods based on the data:

Standard	q_{rule} [MN/m]	Return period	
		per days	annual
ISO 19906	5.9	$2.7 \cdot 10^6$ years	$3.1 \cdot 10^7$ years
IASC PC	2.4	5.5 years	62.5 years
FSICR	0.8	4 days	-
CSR	0.6	2 days	-

CONCLUSION

The main points brought forward by the analysis:

- The design point (or limit state) definitions are incomplete in all standards;
- The (interpreted) definitions leave quite some room for interpretation;
- The probabilistic definitions are somewhat loose;
- Ship and offshore structures rules give very different characteristic actions;
  and
- Validation of design is a challenge as longer time series of measurements does not exist.

Limit state design and methodologies in ice class rules for ships and standards for Arctic offshore structures

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Preprint

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Limit State Design and Methodologies in Ice Class Rules for Ships and Standards for Arctic Offshore Structures

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ABSTRACT: Sea ice poses hazards to ships and offshore structures which create challenges for designers. Designers have to take into account the ice actions resulting from the various ice conditions, interactions and operational scenarios which may occur during the lifetime of structures. In this paper the methodologies used in ship and offshore structure standards, such as the Finnish Swedish Ice Class Rules, International Association of Classification Societies Polar Class rules and ISO 19906, are considered with respect to Limit State Design principles and design air temperature definitions. The paper presents an overview of the design approach used in these standards and especially the criteria for the structural limit used. Case study examples based on ship measurements in the Baltic, as well as design air temperature data from the Russian Arctic, are used to highlight the different approaches and their implementation. The analysis shows clearly that explicit design ice action and structural limit are required for the rational basis in the future development of Limit State definitions in standards accounting for ice actions.

KEY WORDS: Limit State Design; Ice Class Rules; Arctic Offshore Structures; Ice Loads; Structural Response.

1 INTRODUCTION

Codes, standards, classification rules, and regulations provide the framework to achieve a safety level in the design and construction of floating offshore structures and ships (these codes etc. are hereinafter collectively referred to as ‘standards’). A particular aspect of these standards is that they address the required level of safety by defining a limit state. This limit state contains definition both of the resistance, *R* (strength) of the structure and actions, *Q* (loading). It is the aim of this paper to analyse the limit state definitions adopted in the different offshore and ship standards with emphasis for those applicable to low temperatures and ice covered regions.

Standards for Arctic shipping or offshore structures must naturally include description of the ice actions resulting from operating in an environment containing sea ice. Sea ice goes hand-in-hand with cold weather and thus also air temperature must be addressed in the design. For ship design, standards for ships navigating in ice are predominantly based on the Finnish-Swedish Ice Class Rules (FSICR 2010) and the International Association of Classification Societies (IACS) Unified Requirements for Polar Class Ships (IACS PC rules, IACS 2016). These are called ice class rules as both contain definition of several ice classes intended for various environmental severities. For offshore structures, the standards used include International Standards Organisation (ISO) 19900 (ISO 2014) and 19904-1 (ISO 2010a) for floating structures and ISO 19906 (ISO 2010b) for Arctic offshore structures, as well as classification society requirements, such as DNVGL-OS-C101 (DNVGL 2015a). All of these standards incorporate the elements of the design concepts like the description of the actions, calculation models for scantlings and actions to be used, and the structural analyses and response criteria.

In this paper we investigate the similarities and differences between the methodologies of the Arctic offshore standards and the ice class rules, mainly through two examples. The aim is to see if there may be any potential design compatibility. This potential compatibility is especially important in sections of offshore standards describing floating structures, as here a reference is made to the application of ship ice class rules (Section 13.5.1 in ISO 19906).

In this paper the different limit state definitions used in the standards are outlined. The methods, ice actions and structural response used in ship design are also presented based on the ice class rules and compared with those used for offshore floating units. In particular, the aim of the standards is to specifically ensure that the design provides structural integrity, as well as ensuring survival in extreme and accidental events. Thus, the definition of the design point, or Limit State, is investigated in the particular case of first-year ice. The impact of different structural response criteria in association with ice loads applicable to floating structures is demonstrated. This action case was selected as an example owing to the availability of a large data set from full-scale ship measurements of ice actions. The other example deals with design air temperatures using data from the Russian Arctic to illustrate the compatibility (or lack thereof) between design methods for ships and offshore structures.

Where applicable in this paper and to highlight discussion given, direct quotes from standards are written in *italics* and referenced to the relevant section numbers. It should be noted that the standards are frequently updated. Thus the paragraphs that are discussed here may be changed somewhat. For example, ISO 19906 is under review at the moment. however the DIS version of ISO 19906 available at the present time of writing (June 2017) does not indicate significant changes to the principles discussed here but these are highlighted where relevant in the paper.

2. PARADIGM FOR THE LIMIT STATE DESIGN

As there exist several different views on what constitutes a Limit State and how this limit state design is applied, a short description is given here on how the authors understand the definitions of various concepts. Also a brief interpretation of the situation in ship and offshore structure standards is given.

The definition of a limit state requires definition of the structural capability, usually called resistance, and the external and internal actions, such as permanent actions (weight induced), variable (loading condition induced) and environmental actions (ice, wave, wind etc. induced). The actions, especially the environmental actions are typically stochastic and thus a frequency or a probability level must be associated with the definition of actions. Allocation of a probability to resistance is much more difficult task and this is made in neither the offshore structure nor ship standards. In general, the offshore standards explicitly include this structural limit state definition, while ship standards are usually prescriptive i.e. giving just equations for scantlings. These definitions are considered and analysed in some detail later in this paper. In some earlier publications authors used the Limit State synonymously with the term Design Point.

In the limit state design the structure resistance must be greater than the actions; Limit State is then the borderline when resistance is equal to actions. DNVGL (2015a) defines the Limit State as 'A state beyond which the structure no longer satisfies the requirement'. This leads to a principle called Load and Resistance Factor Design (LRFD). In this method the uncertainties in resistance and actions are separated, the situation can be illustrated with the well known diagram, see Figure 1. The LRFD states that:

$$\sum_i Q_i = \sum_i \gamma_f F_i \leq R_d = R_r / \gamma_R \quad (1)$$

where the factors γ_f and γ_R are called action and resistance factors, respectively. Q refers actions, F different loads and R the (structural) resistance. The subscript 'r' refers to representative value which is given at the set probability level and 'd' to design value. The sum in the actions side of the equation implies that load combinations must be considered. LRFD is often applied in the offshore structure standards.

An alternative for LRFD is the classic Working Stress Design (WSD) or Allowable Stress Design (ASD). Here the stresses created by the actions must be less than a limit stress multiplied by a safety factor φ:

$$Q \leq \varphi R \quad (2)$$

Typically ship standards can be assumed to be based on this kind of approach, even if this is not always explicitly stated.

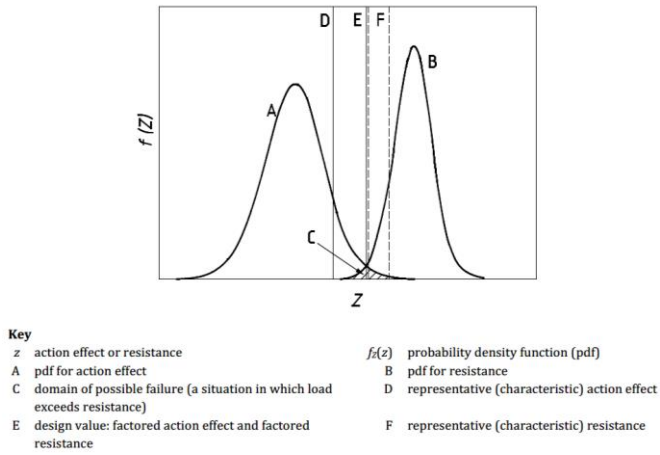

Fig. 1. Distribution of resistance and actions (ISO 19906: Fig. A.7-1).

3. STANDARDS FOR OFFSHORE STRUCTURES

The following provides an outline of the applicable standards for Arctic offshore structures with particular reference to the principles with respect to the ice actions and limit states approaches they adopt.

3.1 Ice Interaction Scenario

For any structure, it is generally necessary to consider and identify several distinct design situations in sufficient number to ensure that the critical action combinations for all main load-bearing structural components are evaluated. These situations are driven by the physical environment and operational characteristics, and are different for both offshore structures and ships mainly because ships operate worldwide and must thus be assumed to encounter all kinds of environments while offshore structures often remain stationary on location.

In offshore standards the design situations result from the intended use of the floating structure in conjunction with the environmental conditions affecting the floating structure's behaviour. Specifically, for Ultimate Limit State (ULS) conditions, representative metocean actions are to be established with the most onerous metocean action effects with return period of 100 years, and the check is normally performed by carrying out a linear elastic response analysis of the structure to determine stresses or stress resultants (moments, deflections). Crucial in evaluating the actions is to be able to derive combined actions (see 19900:9.2.3 and 19904-1:7.8).

For limit state design in ISO 19906, the environmental ice action is a key parameter and is characterised by a given return period and may be related to frequency statistics; medium term environmental conditions for Serviceability Limit State (SLS), and long term distributions for ULS and Accidental Limit State (ALS), see also Table 1. Specifically for ice design, the ice action is related to ice type, ice morphology, ice movement, and ice properties. Here particular emphasis is made for the

information required to characterise site-specific ice criteria for the location of the structure.

The design of offshore structures in ISO 19906 includes global ice actions, relating to the overall integrity of the structure, foundation and stationkeeping system, and local ice actions for specific parts of the structure. The ice actions are determined from a set of interactions between the ice and the structure, and in ISO 19906 these appear to be called 'ice action scenarios' and/or 'design ice actions'. The difference in these is somewhat ambiguous as Extreme Level Ice Event (ELIE) and Abnormal-Level Ice Event (ALIE) are to be *'…determined for each relevant ice loading scenario and when several ice and operational scenarios are relevant for a particular structure, those resulting in the largest ice actions for each limit state are to be considered in the design*' (19906:7.2.4). Indeed, ISO 19906 provides quite an expansive description list of some of the global ice actions to be considered.

Not all of these aspects are applicable for either floating structures or for local ice loads application. In this respect, ISO 19906 states that for steel structures, local actions are to apply to the design of plates, stiffeners, frames and bulkheads and the design contact areas considered based on the local structural configuration, including frame spacing and plate thickness (19906:8.2.5). And the size and placement of the local contact areas (load patches) is to be selected to ensure that the most critical cases are addressed. This statement is at variance with a prescriptive definition of contact areas in section A.8.2.5.1. However, in this aspect ISO 19906 also notes *'…where the global action on the hull is limited by the capacity of the stationkeeping system, the unfactored failure resistance of the stationkeeping system shall be used for the calculation of actions on the hull*' (19906:13.4.4) and referring to ISO 19904-1. This aspect of defining the local design load patch is clearly an area of research and development and future clarification.

3.2 Limit State Definitions

According to ISO 19900, structural design is performed with reference to a specified set of limit states which are divided into four categories, as given in Table 1. For each Limit State, design actions are to be identified, and following this, appropriate calculation models are to be established which address each of the design actions. In this paper we focus on the ULS, SLS and ALS for ice actions for offshore structures, and do not consider the Fatigue Limit State (FLS) further as fatigue involves slightly different action aspects to those we are considering here. It should be noted here that referring to standards, the year of editions should be mentioned as the standards are being frequently updated. Thus in the latest draft version of ISO 19906, the definitions in Table 1 are somewhat edited (see the DIS version, ISO 2017). Here, however, the focus is on how precisely the 'limit' in the Limit State definition is given especially for local steel structures, and these are not changed.

ISO 19906 has been developed with the intention of giving requirements for structures to be capable of resisting ice, and is based on available knowledge and experience on the physical environment from offshore activities in arctic and cold regions. The standard contains requirements for the environmental conditions, ice actions and foundation design for fixed and floating structures. The standard also addresses issues such as topsides winterisation, and escape, evacuation and rescue (EER). For application to floating offshore structures, the standard also references ISO 19900 *General Requirements for Offshore Structures* and ISO 19904-1 *Floating Offshore Structures* which are to be used jointly.

ISO 19906 requires the methodology of Limit States to be applied. However, the explicit form of limit states is modified from that presented in ISO 19900 and ISO 19904-1, see Table 2. Here a distinction may be noted that in ISO 19906 there is greater emphasis and use of 'abnormal' rather than 'accidental' in the ALS definition, which may be attributed to the reference to abnormal environmental events, i.e. ice actions. In addition, specific terms are presented for ice actions as extreme-level ice event (ELIE) for ULS and abnormal-level ice event (ALIE) for ALS associated with a prescribed probability. Serviceability limit states (SLS) are defined rather nebulously *'only assigned quantitative reliability targets under specific circumstances*' (19906:7.1.5).

The difference between ULS and ALS and the related actions ELIE and ALIE is not necessarily clear. What is more severe, 'extreme' or 'abnormal'? The ISO 19906 (paragraphs 7.2.2.3 and 7.2.2.4) define the return periods of ELIE and ALIE; these are 100 years and 10 000 years, respectively, see Table 2. Clearly making 'abnormal' more severe than 'extreme'.

ISO 19906 requires the structure to be assessed for strength and stiffness with respect to ULS. This is in agreement with ISO 19904-1 where the limit state categories used in the structural design of a floating platform for ULS involve checking the floating structure's strength to resist extreme actions. For 19904-1 in open water conditions, this is typically achieved by ultimate hull girder calculations in structural design requirements of offshore ships, e.g. as outlined in DNVGL OS-C102 (DNVGL 2015b).

Table 1. Limit state definitions according to ISO 19906:7.2.2.1 (edited slightly for conciseness).

Limit State	Ultimate Limit States (ULS)	Serviceability Limit States (SLS)	Fatigue Limit States (FLS)	Accidental Limit States (ALS)
Definition	Correspond to resistance to extreme applied actions	Correspond to criteria governing normal functional use	Correspond to the accumulated effect of repetitive actions	Correspond to accidental events and abnormal environmental events

For ALS, ISO 19906 requires adequate reserve capacity and energy dissipation capability. In ISO 19904-1 the conditions to maintain global integrity during an accidental event for ALS are elaborated, such as fire/blast, collisions, compartmental flooding, mooring line failure, dropped objects, and fluid impacts such as green water or slamming. Furthermore, the structure is to be designed so that '*if structural damage does occur, the damaged structure (possibly with temporary repairs, as applicable) is able to resist action combinations appropriate to these design situations without suffering extensive failure, free drifting, capsizing or sinking, and without causing extensive harm to the environment'* (19904-1:5.5.2). It is noted in this context that ALS in ISO 19906 appears predominantly focused on ice action events with probability 10^{-4}, such as iceberg impacts.

A further stipulation in ISO 19906 is the requirement that action effects arising from factored action combinations are not to exceed factored resistances. These combination or companion actions, including conditions such as wave and ice, are not commented further here and are subject of other focused studies.

Table 2. Limit state definitions according to ISO 19906:7.1 and 7.2 (modified slightly for concision).

Limit State	Description	
Ultimate Limit States (ULS)	The ULS requirement ensures that no significant structural damage occurs during the design service life of the structure. The ULS design condition for ice shall be the extreme-level ice event (ELIE). Both local and global actions shall be considered (7.2.1.2)	For ULS, the design procedures shall be based primarily on linear elastic methods of structural analysis. Some localized inelastic behaviour is acceptable. The design action arising from ELIE shall be determined based on an annual probability of exceedance not greater than 10^{-2} (7.2.2.3).
Serviceability Limit States (SLS)	Exceedance of SLS results in the loss of capability of a structure to perform adequately under normal use. The specification of actions for SLS is generally the owner's responsibility ... (7.2.1.3).	For SLS, the structure should satisfy the requirements of this International Standard when structural members are subjected to localized ice damage. Examples include: concrete cracking and shrinkage; loss of concrete cover due to ice abrasion; removal of paint and removal of corroded steel; and localized damage as a consequence of vibrations induced by ice. The structure shall also satisfy serviceability limits for vibrations and deflections. Unless the owner has specified otherwise, the design action of the SLIE used for SLS shall be determined based on an annual probability of exceedance not greater than 10^{-1} (7.2.2.2).
Accidental Limit States (ALS)	The ALS requirement is intended to ensure that the structure and foundation have sufficient reserve strength, displacement or energy dissipation capacity to sustain large actions and other action effects in the inelastic region without complete loss of integrity. Some structural damage can be allowed for ALS. The ALS design condition for ice shall be the ALIE. Both local and global actions shall be considered (7.2.1.5).	For ALS, non-linear methods may be used. Structural components are allowed to behave plastically, and foundation piles are allowed to reach axial capacity or develop plastic behaviour. The design is based on a combination of static reserve strength, ductility and energy dissipation to resist the ALIE conditions. The design action arising from the ALIE shall be determined based on an annual probability of exceedance related to the exposure level being not greater than 10^{-4} or shall be derived from events with an annual probability of occurrence not greater than 10^{-4}. Iceberg and ice island impact events with an annual probability of occurrence between 10^{-4} and 10^{-5} and with high consequences should be considered for ALIE to ensure adequate reliability (7.2.2.4).

The description of Limit States shows that the 'limit' is not defined with any exactness. For example ULS states that '*Some localized inelastic behaviour is acceptable' (19906:7.2.2.3).* How much deformation is 'some'? What is meant with 'localized'? Alternatives for a more exact definition for the structural limit are described in the second example below.

3.3 Example Applications

Two examples of the application of ISO 19906 are given to illustrate the limit state definitions. The first example (design air temperatures) is selected as it illustrates the different definitions for a seemingly clear parameter. The second example is selected to illustrate the different limit state definitions where a large measurement database is available. It is not common to have this long (seven consecutive winters) database and this way the relationship between the return period and resulting scantling (here plate thickness) can be investigated.

<u>Design air temperature</u>

The first example examines the definition of the design air temperature, denoted as T_D. The design air temperature in ISO 19906 is specified by: '*The lowest anticipated service temperature (LAST) shall be defined in accordance with 3.48'* (19906: 6.3.1). The referenced paragraph states: '*LAST: Minimum hourly average extreme-level (EL) air temperature'*. This point is further emphasised in 11.9.1: '*...the LAST value used for material selection and testing should be defined in accordance with this International Standard...'* and also in topsides chapter 15.1.1.2: '*Structural materials should be selected based on the requirements of 11.9 and the ... LAST'*.

Thus the design temperature is the lowest annual temperature based on a return period of 100 years.

The calculation of LAST is, however, slightly problematic. Should the statistics used to derive the $T_{100\ years}$ include all hourly measured temperatures (8760 points per year)? Or should each day be analysed individually and then the minimum given from each day selected as the basis for LAST? Further, the height of the temperature measurement point is not specified.

Local steel structures

The second example is the design of local steel grillages as here a direct comparison with ship standards can be made, because in the ISO 19906 there is a reference to using ship standards: '*The designer may utilize the appropriate formulations in guidelines for ice-strengthened vessels of a recognized classification society'* (19906: 13.5.1). This case is discussed in detail in later sections and the focus here is on the definition of limiting response (resistance) even if the definition of the representative ice action for plate grillages also is less than straightforward.

The following definitions for the structural limit can be collected from the Limit State definitions in ISO 19906:

SLS: 'subjected to localised ice damage' (7.2.2.2), '[without] the loss of capability to perform adequately under normal use' (7.2.1.3);

ULS: 'no significant structural damage' (7.2.1.2), 'design procedures shall be based primarily on linear elastic methods' (7.2.2.3), 'both local and global actions shall be considered' (7.2.1.2);

ALS: 'structural components are allowed to behave plastically' (7.2.2.4), 'without complete loss of integrity' (7.2.1.5).

These definitions leave the designer quite much to interpret both quantitatively (how deep a dent is to be considered as not 'significant'?) and qualitatively (should the plastic design consider only the plating or also the primary frames, secondary frames and bulkheads, as the ULS definitions mention both plasticity and local and global actions?).

There is a distinction in ISO 19906 between the 'global' ice pressure and 'local' ice pressure used to design local structures. The global ice pressure is given for vertical structures (19906: A.8.2.4.3) in two formulations. The first one is based on measurements in different seas and is given as (19906:A.8.2.4.3.3):

$$p_G = C_R \left(\frac{h}{h_1}\right)^n \left(\frac{w}{h}\right)^m \tag{3}$$

where w is structure width, h ice thickness and constants h_1 = 1 m, n = -0.5 + h/5 (h < 1 m) and m = -0.16 (note that there is a suggestion for the updated 19906 to have a small correction to this for lower aspect ratios). The constant C_R accounts for ice strength and also statistics. Its value is stated to be 2.8 with a return period of 100 years (ULS case). Note that this equation is to be amended in the updated version of 19906, but the following conclusions are still valid. Apart from (3), there is given a pressure – area relationship from ship ramming tests (19906: A.8.2.4.3.5). The shape of the structure, be it the cross section shape or inclination, is not taken into account in these formulations.

The design local ice load patch for local transverse frames is assumed to be 0.4h in height with width w_L = s for plate and 4s for transverse frames, where s is the frame spacing. Then the force on the design area is given as (in MN):

$$F_L = 3.72 \cdot \sqrt{0.4h} \cdot w_L = 2.35 \cdot \sqrt{h} \cdot w_L$$

and the pressure (in MPa, h > 0.35m):

$$p_F = 5.875 h^{-0.5}$$

The ice thickness to be used corresponds to ULS case, i.e. ice thickness corresponds to a return period of 100 years. The difference in the resulting scantlings is analysed later in this paper.

4. STANDARDS FOR SHIPS

The main design standards for ships are presented by the International Maritime Organisation (IMO) and International Association of Classification Societies (IACS) members' rules which establish the minimum requirements to mitigate the risks of major hull structural failure. In particular, the IACS Common Structural Rules (CSR) apply to the hull structures of oil tankers and bulk carriers, and provide rules in terms of loads, structural capacity models and assessment criteria and also construction and in-service aspects (DNVGL 2015b). They are also based on the principles of limit state design where each structural element is evaluated with respect to possible failure modes related to the design scenarios identified, and the limit load for the structural element is found as the minimum limit load resulting from all the relevant limit states. It is therefore a useful reference in this study.

The IACS CSR, however, do not account for ice navigation of ships. Provisions for these ships are contained in the IMO International Code for Ships Operating in Polar Waters (Polar Code) (IMO 2014). The Polar Code has been developed to supplement existing IMO instruments in order to increase the safety of ships' operation in the Arctic. The provisions in the Polar Code consist of an overall goal, functional requirements and regulations. For ice strengthened ships the goal for ship structure is for '*...scantlings of the structure retain their structural integrity based on global and local response due to environmental loads and conditions*' (Polar Code: Part I-A;3.1), which is similar to the functional requirement where '*…the structure of the ship shall be designed to resist both global and local structural loads anticipated under the foreseen ice conditions*'(Polar Code: Part I-A;3.2.2). The Polar Code then references standards acceptable to the Organization (i.e. IMO) or other standards offering an equivalent level of safety, with footnote to the IACS URI Requirements concerning IACS Polar Classes, which are discussed in detail in the subsequent sections. For structural integrity, the Polar Code also includes stability

provisions in damaged conditions which require the ship to be able to withstand flooding resulting from hull penetration due to ice impact (Section 4.3.2). Although not stated explicitly, this scenario may be considered as a potential ALS design situation for ice class ships.

The IACS Unified Requirements for Polar Ships (hereinafter referred to as IACS PC rules) provide technical requirements for design of ice strengthened ships. These rules consist of three sets of unified requirements; UR I1 (Polar Class Descriptions and Application), UR I2 (Structural Requirements for Polar Class Ships), and UR I3 (Machinery Requirements for Polar Class Ships). The requirements are framed on seven polar classes which are described in terms of nominal ice conditions with the intention to provide a relatively smooth increase in requirements, see Table 4. The description of the ice classes were deliberately kept general due to the considerable variability of ice conditions (IMO 2011). The two lowest IACS Polar Classes, PC6 and PC7, are recognised as nominally equivalent to the Finnish-Swedish ice classes, IA Super and IA.

The Finnish Ice Class Rules (FSICR) are considered an industry standard for ships navigating in sea areas where there is only first-year ice. Four ice classes are defined and are, in order of strength from high to low; IA Super, IA, IB, and IC. The strength level in each ice class corresponds roughly to the loading from a certain level of ice thickness; 1.0 m for IA Super down to 0.4 m for IC.

The FSICR have a long development history; see (Riska & Kämäräinen 2011). Results from ice damage surveys have been incorporated into the ice class rules, such as Johansson (1967). Here for the first time feedback from designs gave an idea of the ice action level. The next revision from the 1971 version of the FSICR changed the ice load height, based on extensive ice load measurements on merchant ships and icebreakers, see for example, Vuorio et al. (1979). The idea here was that ice is similar, even if the thickness is somewhat greater, and thus the ice pressure, being an ice material constant, is the same for each ice class; but as the ice thickness related to each ice class is different, the load heights are consequently different for each ice class. While the load height changed, the line load magnitude q (i.e. ice pressure times load height, $q = p \cdot h$) determined from the ice damage data, remained constant in the consecutive updates of the rules. This idea of the ice load and ice load height is illustrated in Figure 2.

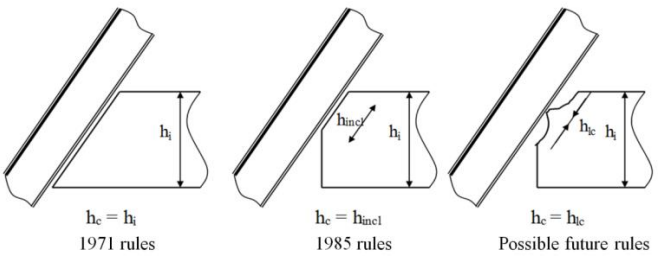

Fig. 2. Load height h_c concept with constant value of the line load q at about q = 2 MN/m. hi is the level ice thickness and h_{incl} and h_{lc} refer to particular load heights.

4.1 Ice Action Scenario

During the development of the IACS PC rules, different ship-ice interaction scenarios were identified which were considered potentially relevant for structural design. The load formulation for the IACS PC rules strength requirements was based on a set of ship-ice interaction scenarios (McCallum 1996) which are illustrated in Figure 3. Figure 3 is provided for illustration only, and the glancing impact that was the basis for local ice action formulation can occur in open pack as well as in level ice. From this study, and as in the current IACS PC rules, two ice interaction scenarios are considered to be governing, namely a ramming scenario on multi-year ice and a glancing impact scenario where ice is acting on the other side of the ship bow. The ramming scenario where the ship bow collides with a large multi-year ice ridge is considered to be governing for the longitudinal (global) strength. The other, glancing impact scenario, where the ship is assumed to strike an ice edge of infinite mass with the bow shoulder, is considered to be dimensioning for the structural design of the bow (and used as a basis for the other parts of the hull structure) (IACS 2016c). During IACS PC rule development, the load patch was modified from the actual one in the glancing impact so that only area was considered (and not the shape).The IACS PC rules are founded on a glancing impact to the bow, and are therefore implicitly based on ships breaking ice independently; that is, without escort. However, the mode of sailing is usually related to the operational ethos of maritime administrations, which provide icebreakers to operate in that region. For example, the powering requirements given in the FSICR are based on a continuity of trade principle, so there are always icebreakers present to make and maintain ice channels, and escort ships when necessary. Thus, each rule set has been calibrated for the relevant modes of operation. Should a ship with icebreaking capability desire to act independently, then IACS PC rules are applicable but the FSICR may not be particularly applicable, as they are based on navigation with icebreaker escort. Similar caveat may be stated concerning ice management in floating offshore structures.

SHIP/ICE INTERACTION SCENARIO BY REGION/SEASON
ARCTIC - EASTERN HEMISPHERE
RUSSIAN

SHIP/ICE SCENARIO - WINTER		REGIONS						
		Bering	*Chuckchi Sea*	*East Siberian*	*Kara*	*Laptev*	*Northern Barents*	*Southern Barents*
Level Ice	1. Proceeding continuously ahead in level ice	3	3	3	2	2	2	3
	2. Going astern or manoevuring in level ice	3	3	3	2	2	2	3
Ramming	1. Ramming in level ice	2	2	4	4	4	2	3
	2. Ramming ridges	4	4	4	4	4	4	3
	3. Ramming and manoeuvring in ridges and hummock field	5	5	5	5	5	4	3
Open Pack	1. Glancing impact on a floe	0	0	0	0	0	0	4
	2. Reflected impact from one floe to another	0	0	0	0	0	0	4
	3. Striking a floe on the quarter during a turn	0	0	0	0	0	0	4
Wedging	1. Wedging between two floes (A) or a narrow lead (B)	2	2	4	4	4	3	2
	2. Ship enters a narrow lead; rides up on one side, falls sideways	2	2	4	4	4	3	2
Dangerous	1. Striking a concealed multi-year flow or growler	3	3	3	2	3	4	2
	2. Striking the underwater projection of an iceberg	0	0	0	2	1	3	3
	3. Proceeding in open pack at speed-making contact with heavy ice	0	0	0	2	1	3	3
	4. Ramming a growler among whitecap waves	0	0	0	0	0	0	2
	5. Ramming (A) shoulder impact (B) iceberg, floeberg or ice island	0	0	0	2	1	3	3
Icebreaker	1. Striking floes in the track of an icebreaker	5	5	5	5	5	5	2
	2. Heavy floes pushed against an escorted ship by an icebreaker	5	5	5	5	5	5	2
	3. Escorted ship following an icebreaker unable to negotiate turns in track	5	5	5	5	5	5	1

Legend for encounter frequencies: 0 – nil, 1-very low, 2-low, 3-moderate, 4-high, 5-very high

Fig. 3. Example of ship/ice interaction scenarios (McCallum 1996).

IACS UR I2 has been updated for application to icebreakers. The basis is to use the same scenario of a glancing impact, but increase the hull area factors to account for the general loading increase due to a more aggressive operational profile (IACS 2016c). The logic of this is reasonable, if somewhat fuzzy, since aggressive manoeuvres would imply a ramming scenario or increase in frequency of events. Further, the patch loading could also change in the horizontal and vertical extensions.

Contrary to the IACS PC rules, the design point for each ice class in the FSICR is a collision with a channel edge (as ships are escorted in the worst ice conditions) or ridges with a consolidated layer . It should be noted that this design scenario does not state ship speed as it is considered that no speed restrictions should exist, as this would handicap much of the navigation in ice. Speed is an important local strength issue though, particularly for application of ship rules which incorporate impact speed into the ice load formulations, whereas for offshore structures this is likely to be limited to ice drift speed. However, it is still somewhat unclear, which ship-ice interaction scenario causes the highest ice action and whether this is speed dependent. See the example in Figure 4 below. These results are from ice load measurement with an icebreaking ferry carried out in Finnish southern archipelago. The ferry was tested in ship channel and in level ice and shows variations in structural response with speed. Important consideration here is the distinction between local and global actions. Ship rules implicitly consider only local ice action (with the addition of longitudinal strength considered in IACS PC rules) whereas offshore standards distinguish between 'local' and 'global'.

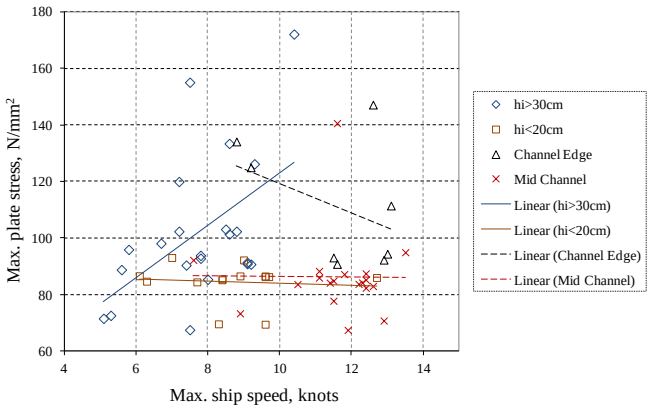

Fig. 4. Measured maxima of plate stress vs. ship speed for different ice thicknesses (h_i) and ice conditions (Bridges et al 2011).

Another consideration in the design situations for ice class vessels is that Baltic ships, and those complying with the FSICR, compete with open water ships during a large part of the year, thus the ice strengthening and ice performance must not have a great effect on their competitiveness, vis-à-vis the open water ships. This has led to the present balance between the required ice capability of merchant ships and the number of icebreakers escorting them, and the safety level of winter navigation. This is naturally not the case for Arctic offshore structures.

The load patch for the design ice action should also be noted (see the discussion of this for offshore structures). The FSICR defines the load patch height which actually is the class factor, see Figure 2. Here a reduction of height relative to ice thickness occurs due to inclined ship hull whereas in ISO 19906 the 40% reduction in the load height versus ice thickness is due to non-uniform ice pressure distribution. For IACS PC rules the load patch dimensions are calculated and mainly the load patch height is used in the design equations.

4.2 Limit State Definitions

The definitions of the Limit State in ship standards are practically non-existent, although some careful reading of different sources yields some understanding on what kind of Limit State is used. This understanding is described here.

The IACS CSR provides structural response criteria for the assessment of the scantlings, and are therefore a useful reference standard. The acceptance criteria are calibrated for the various requirements for combinations of static and dynamic load effects. For example, the acceptance criteria are categorised into three acceptance criteria (AC) sets, see Table 3. The specific acceptance criteria set that is applied in the rule requirements is dependent on the probability level of the characteristic combined load. For instance the acceptance criteria set applied for the static design load combinations has lower allowable stress for such loads than that for an extreme load to take into account effects of repeated yield, allowance for some dynamics, and margins for some selected limited operational mistakes.

In the IACS PC rules the general design scenarios are described with some detail, see Table 4. The IACS PC rules use for the resistance an implicit definition: '*Local frames…are to be dimensioned such that the combined effects of shear and bending do not exceed the plastic strength of the member'* (I2.6.1). Presumably this applies also to plating. There is no mention in the standards about what is the frequency basis (return period) of the ice action. In the technical background material for the IACS rules there is mentioned that the ice actions can be considered to be based on an annual probability level of 1.0 (return period one year) (IACS 2016c).

Table 3. Illustration of principal acceptance criteria from extract of IACS CSR rule requirements (IACS 2007)

Acceptance Criteria set no.	Plate panels and local support members	Primary support members	Hull girder members
AC1	70-80% of yield stress	70-75% of yield stress	75% of yield stress
AC2	90-100% of yield stress	85% of yield stress	90-100% of yield stress
AC3	Plastic criteria	Plastic criteria	NA

Table 4. The definition of different PC ice classes (IACS 2016).

Polar class	Ice Description (based on WMO Sea Ice Nomenclature)
PC 1	Year-round operation in all Polar waters
PC 2	Year-round operation in moderate multi-year ice conditions
PC 3	Year-round operation in second-year ice which may include multi-year ice inclusions
PC 4	Year-round operation in thick first-year ice which may include old ice inclusions
PC 5	Year-round operation in medium first-year ice which may include old ice inclusions
PC 6	Summer/autumn operation in medium first-year ice which may include old ice inclusions
PC 7	Summer/autumn operation in thin first-year ice which may include old ice inclusions

The definitions of the ice classes in FSICR are similarly short. The definition of an ice class in FSICR include the expected operating mode as well as an estimate of the maximum allowable level ice thickness (FSICR §1.1 and §4.2.1):

1. Ice class IA Super; ships with such structure, engine output and other properties that they are normally capable of navigating in difficult ice conditions without the assistance of icebreakers. Maximum level ice thickness h_0 = 1.0 m;
2. Ice class IA; ships with such structure, engine output and other properties that they are capable of navigating in difficult ice conditions, with the assistance of icebreakers when necessary. Maximum level ice thickness h_0 = 0.8 m;
3. Ice class IB; ships with such structure, engine output and other properties that they are capable of navigating in moderate ice conditions, with the assistance of icebreakers when necessary. Maximum level ice thickness h_0 = 0.6 m;
4. Ice class IC; ships with such structure, engine output and other properties that they are capable of navigating in light ice conditions, with the assistance of icebreakers when necessary. Maximum level ice thickness h_0 = 0.4.m.

The Finnish maritime authorities have published Guidelines for application of the FSICR (FTSA 2011). An extract from them gives; '*Ice loads given in the Rules have been determined based on measurements on ships that sail in the Baltic Sea in winter. The situation where a ship is stuck in compressive and/or moving ice and large ice forces are acting on the parallel midbody is not considered in the rules. It is assumed that icebreaker assistance is available in such cases so that there is no time for a serious compressive situation to develop. However, according to the experience of the Administrations, vessels strengthened to ice classes IA and IA Super rarely get damaged in compressive ice situations. During recent years, ice damages on the midbody of ships with ice class IC have been observed'(4.1.2).* These Guidelines also suggest that the elastic limit is used as the structural limit in the FSICR. It is still unclear what the return period of reaching this limit is. This question is analysed later in this paper.

4.3 Example Applications

Similar to the previous section for the offshore structures, the same two application examples are investigated for ships to illustrate the methodologies and approaches used in the standards.

<u>Design air temperature</u>
There are two primary sources for the definition of design air temperature used in ship design; a requirement from IACS UR S6 (IACS 2016b), and that of the IMO Polar Code (IMO 2014). Both of these have a similar basis, although are slightly different. The IACS UR S6 defines the ship design temperature as: '*The design temperature t_D is to be taken as the lowest mean daily average air temperature in the area of operation' (S6.3).* The daily low temperature is taken for each day of the year over a minimum 10 year period. This definition is then further elaborated as being minus 13 degrees for application of the IMO Polar Code. The other design temperature definition for ships is given in the Polar Code: '*Polar Service Temperature (PST) means a temperature ... which shall be set at least 10°C below the lowest Mean Daily Low Temperature (MLDT) for the intended area and season of operation in polar waters'* (Part I-A;1.2.11). A further definition for design temperature often used is the Absolute Minimum which is typically associated with the lowest temperature measured at the chosen location during the data collection period. The application of these definitions is analysed in the next chapter.

<u>Ship shell structure</u>
The limit response in the IACS PC rules allows plastic response whereas FSICR set the structural limit to the yield point. The probability level of the ice action for local ship structures is not stated in the rules, as discussed above.

The ice load characteristics given for the ships are more explicit than in ISO 19906, dealing with much fewer load models. The load calculation in the IACS Polar Ship Rules is somewhat convoluted dealing with force, line force (force per load patch width) and ice pressure. The formula included in

the IACS Polar Ship Rules for bow ice force uses an assumed value for ship speed and ice thickness (which increase with ice class) given as follows:

$$F_i = C_f D^{0.64} \tag{6}$$

where C_f is a factor accounting for hull shape and ice class (for the two lowest classes it is approximately 1.12 and 1.05) and D is ship displacement. Bow ice pressure is given as:

$$P_i = C_p F_i^{0.22} \tag{7}$$

where again the factor C_p depends on hull shape and ice class (for the two lowest classes it is approximately 1.74 and 1.65).

The design bow ice pressure concept in the FSICR differs from the IACS PC model and is given by the following:

$$p = c_d c_a p_0 \tag{8}$$

The ice pressure is the same for all classes (nominal ice pressure is p_0 = 5.6 MPa) and the load height h is the class factor (from 0.35 m for ice class IA Super to 0.22 m for ice class IC). This approach of the same pressure for all ice classes is taken as the material properties of the Baltic ice do not change much through the winter in different Baltic Sea areas even if different ice classes are intended to navigate during different seasons or areas (lower classes only early winter and southern sea areas).

A ship size coefficient for ice pressure is c_d, which is linearly dependent on $k = \sqrt{P_D \Delta}$ where P_D is propulsion power and Δ ship displacement. Typically c_d is about 0.7. The FSICR also use a factor to define the ice pressure as a coefficient dependent on the load length, c_a. Each structural member has an associated load length l_a which is the length of the load that influences the response (stress) in the member. The load length coefficient is defined as:

$$c_a = \sqrt{\frac{l_0}{l_a}}, \quad \text{max 1.0 and min 0.35}$$

where the reference length is l_0 = 0.6 m. In principle, each structural member should be designed by trying all load lengths and then selecting the design case to be the length that gives the maximum response. There is no need to do this calculation, as the load lengths are given in the rules, as shown in Table 5.

Table 5. Load lengths associated with different structural members (FSICR 2010).

Structural member	Type of framing	Design load length l_a [m]
Shell plating	Transverse	Frame spacing
	Longitudinal	1.7 · frame spacing
Frames	Transverse	Frame spacing
	Longitudinal	Span of frame
Ice stringer		Span of stringer
Web frame		2 · web frame spacing

The total ice load for each structural member is then taken as the ice pressure times a load length l_a that depends on the width (horizontal span or spacing) of each structural member and load height that is a class factor.

5. CASE STUDIES

In the following, the two cases described as examples are investigated in practical case studies. The design air temperature issue is analysed based on the air temperature regime in the Russian Arctic, and the design of local shell structure (only plate thickness considered as the frame design is very similar) is investigated based on a set of long term measurements in the Baltic.

5.1 Design Air Temperature

The result of the various definitions for design air temperature are investigated using data from Russian Arctic, specifically from a station on Vaygach Island, with location at Lat(°N) of 70.27 and Lon(°E) of 59.05. The air temperature data was extracted from the NCEP Climate Forecast System Reanalysis (CFSR). It includes 6-hourly temperature measurement from January 1967 to December 2010, see Saha et al. (2010). It may be noted here that hourly data for Arctic locations as required by ISO 19906 is very sparsely available.

Using the dataset, the daily temperatures are plotted versus the date in Figure 5 and based on a statistical analysis versus the return period in Figure 6. It should be noted here that there is much room for interpretation of the temperature data, such as the length of data to use, i.e. which 10 years to use in a 20 year data set, or the number of stations needed to ensure reliability of data.

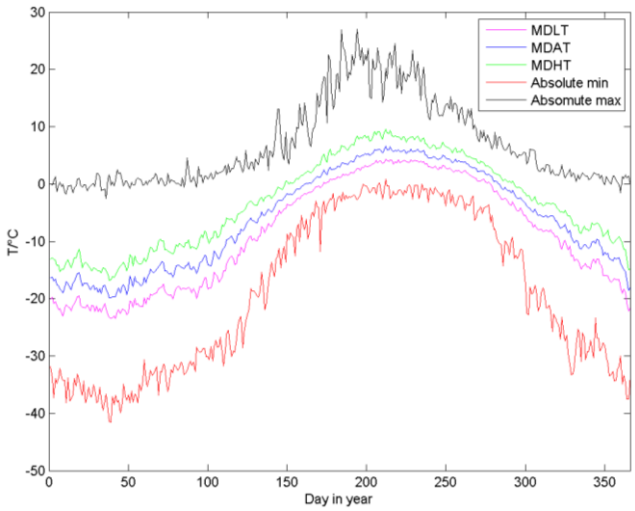

Fig. 5. Example of design temperature values over year based on analysis of daily air temperature data. The temperature definitions used are; MDLT-mean daily lowest temperature, MDAT-mean daily average temperature and MDHT-mean daily highest temperature.

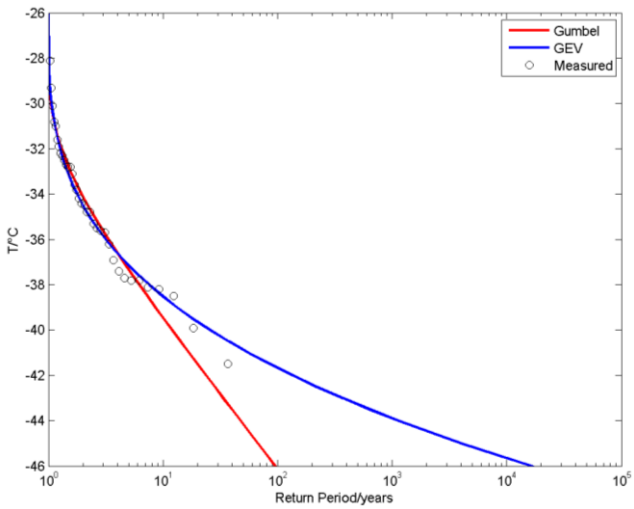

Fig. 6. The temperature observations plotted versus the return period and two statistical distribution fits.

Statistical analysis was carried out on these data points by fitting a Gumbel double exponential distribution and a Generalized Extreme Value distribution (GEV). The cumulative distribution of GEV is of form $\exp(-(1+\beta s)^{-1/\beta})$ where s is the standardized variable, $s = (T-\mu)/\sigma$ where T is temperature and μ its average value and σ standard deviation. These two distributions were selected to highlight the difference obtained when using several distributions. The result is shown in Figure 6. Table 6 presents the results of the analysis using the various design definitions used in ship requirements and in the offshore standards with different return periods, as well as the Absolute Minimum as the reference value.

Table 6. Design air temperature results from CFSR data set.

Design Temperature			°C	Date
Lowest MDLT			-23.44	18 February
Polar Service Temperature (PST)			-33.44	
IACS UR S6 (lowest MDAT)			-19.82	
Absolute Minimum			-41.50	08/02/1979
Return Period (Gumbel) (years)	LAST	*100*	-46.12	
		10 000	-52.93	

Table 6 shows a difference in values with alternative design temperature definitions. It is interesting that temperature corresponding to 100 years return period is relatively close to the Absolute Minimum measured. In the shipping industry a common practise is to use the lowest measured temperature in the operating area as the design air temperature. Thus PST and lowest MDAT seem to be somewhat high and the value used in ISO 19906 (LAST) close to the value used in practise.

5.2 Design of Plating

As an example of applying the offshore and ship standards to design of a steel structure, a case for the northern Baltic is used. The selection of the example is based mainly on the existence of a large full scale data set; the measurements from the ship MT *Kemira*. MT *Kemira* is used here as the ship was instrumented to measure the ice load on one frame as well as the plate and frame stresses continuously during seven winters (1985-91). MT *Kemira* is a chemical carrier that operated between the ports Kokkola and Uusikaupunki in Finland and central European ports.

Main particulars of the ship are:

Ice class	IA Super
L_{pp}	105.0 m
B	17.5 m
Power	3400 kW
Deadweight	5800 dwt

The operational spectrum of MT *Kemira* can be assumed to represent a typical Baltic merchant ship in size and shape, and the years integrate different winters (three severe, two average, and two mild) as well as the ice thickness development throughout each winter. MT *Kemira's* ice load data are described by Kujala (1989), Gyldén and Riska (1989), and Muhonen (1991, 1992). The measurement system did not record the time histories of the signals but, instead, calculated two 12 hourly histograms for each measurement channel. One histogram consists of the regular sampling from each measurement channel with a sampling rate of 100 Hz. The other histogram includes the signal peaks, with each peak identified using the Rayleigh separation for the peaks (with 25% of the peak value as the separator). These histograms were stored after each 12 h period throughout the winter for the years 1985-1991. The data used here are derived from the maxima of each 12 h peak distribution. Only the maxima from periods when the ship was sailing in ice are included. Example of the data and the Gumbel fit are shown in Figure 7. The data allows an investigation of the return periods associated with the rule design values as well as assessment of the structural limit used in the various standards.

Kemira data includes about 200 operation days in ice from seven winters, i.e. about 30 operation days in ice per winter. As the ULS case corresponds to 100 years, the question arises should the extrapolation be to ELIE be based on 100·365 (=36,500) days or taking just the operational days into account, i.e. ELIE would be based on 30·100 (=3,000) days. Here it should also be mentioned that extrapolation from seven years of data to 100 or 10 000 years entails a large uncertainty. The approach taken here is to simply make estimates for comparisons, but this is clearly another factor for future research to clarify.

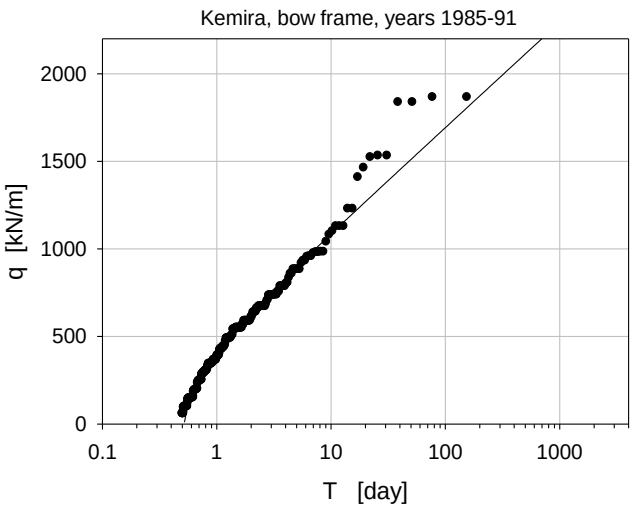

Fig. 7. Measured and estimated (according to Gumbel I pdf) loads versus the return period for bow frame load (Riska & Kämäräinen 2011).

As a design ice action, the line load in horizontal direction, q [MN/m], is used. This quantity is used as it is common to describe frame ice action using this. Ice pressure is obtained by dividing with ice load height as the line load is assumed to correspond to (average) ice pressure times the load height. For the shape of the *Kemira* bow, this load height may be estimated to be about one third of ice thickness; a maximum of about 35 cm. It is clear that the *Kemira* measurements include a sloping structure. The effect of the shape of the indenting body (e.g. ship bow or surface of offshore structure, etc.) may influence the load. The eq. (3) in ISO 19906 is under the heading 'Vertical Structures' (ISO19906: A.8.2.4.3), even if there is under this heading also ship data presented which are obtained from sloping structures. Further, the local loads (ISO 19906: A.8.2.5) do not refer to shape. Ship and offshore structure (and also lighthouses, etc.) have also a widely varying indentation speeds. Thus the comparison of design actions deduced (especially from ISO 19906) should be analysed more thoroughly, but this is a matter of another investigation. Here we neglect these aspects and present just a comparison to permit illustrative relationships.

The design ice action according to the various standards is calculated as follows. Assuming the ELIE ice thickness to be 1m, which is an estimate based on the maximum level ice thickness measured ever in the Baltic, 1.22 m (Seinä et al. 1997). Thus ISO calculation (19906: A.8.2.5.2.3) gives a line load value of 5.9 MN/m, using a load height of 40% of h (19906: A.8.2.5.2.3). The design value in FSICR is 0.8 MN/m for ice class IA Super while the value for IACS PC6 class is about 2.4 MN/m. The latter value is an estimate as the hull shape at the contact influences the action; in this calculation the hull angles for MT *Kemira* have been used to give values similar to the full scale data.

The ELIE ice action value (return period of 36500 days) from measurements using the Gumbel I fit gives 3.3 MN/m. It should be noted that there is a difference in interpreting the 'annual probability level'. Annual maxima could be used and in the case of measurements it corresponds then to 30 days maximum as MT *Kemira* operated in ice only 30 days per year on average during the 7 years measurement period. An alternative is to use a return period of 36500 days corresponding to 100 years. These two cases are included here. Thus if the return period is 100 years (winters), the ice action value is 2.6 MN/m. The data allows also calculation of the return period related to the design value specified in the ship and offshore standards, see Table 7.

Table 7. Ice action values in different standards and corresponding return period, calculated both on daily and annual basis. Denotation 'annual' refers to only a limited number of ice operation days in a year (for MT *Kemira* this is 32 days).

Standard	q_{rule} [MN/m]	Return period	
		per days	annual
ISO 19906	5.9	$2.7 \cdot 10^6$ years	$3.1 \cdot 10^7$ years
IACS PC	2.4	5.5 years	62.5 years
FSICR	0.8	4 days	-

A large variation is clearly observed in the return periods corresponding to the design values derived. The low value (of 4 day return period) in the FSICR is clearly observed in comparison with IACS and ISO 19906. As the reason for this may be in the definition of the structural limit, this is analysed next. The design situation is sketched in Figure 8. The design process shown in this figure can be termed Limit State Design as the right side of the figure shows the ice action versus the return period and left side the relationship between the scantling (here plate thickness) and the ice action for several different limit responses (here different permanent deflections). The lines shown for these are sketches as the relationship is not linear. Two quantities must be pre-selected for design; the return period of the ice action and the allowed (limit) response. The arrows in Figure 8 depict that two different design return periods and different allowable responses can give similar scantlings. For a more detailed description, see Riska (2014). Thus, if the return period is short but the structural limit strict (such as in FSICR), then the resulting scantlings are the same as if the return period longer but the structural limit would allow more response.

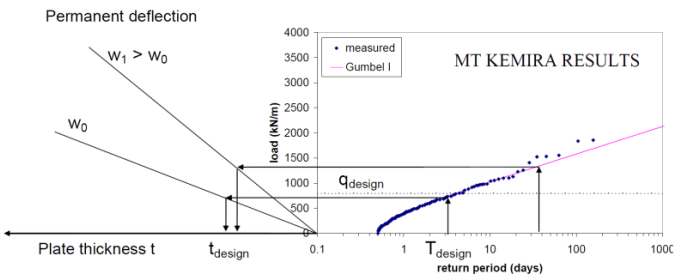

Fig. 8. A sketch of the design point definition, where w_0 and w_1 are simply arbitrary values for illustrative purposes (Riska 2007).

The concept of the Limit State includes a definition of the structural limit and the frequency with which this limit state is reached. The structural limit function $L(Q,R)$ includes

the action Q and the strength of the structure R (determined by scantlings, material properties, and geometry) – the latter is often expressed as the load causing a response up to the selected structural limit. The structural limit in the following analysis will either be stress reaching yield stress somewhere in the structure, full plasticity, i.e at some point in the structure the (bending) stress distribution is fully plastic, or some specified permanent deformation. If the limit is formally denoted as w (being Y, P, or U, respectively, in Table 8) and the structural details (scantlings etc.) as d – this can be a vector, i.e., contain more than one parameter – then the structural limit (which is commonly called 'strength' of the structure) can formally be presented at the limit where $Q = R$ as a functional form:

$$Q = r(w, d)$$

At the same time, plots such as in Figure 7 give the most probable load in a given time, i.e, the return period T, as:

$$Q = s(T)$$

To obtain the relationship $g(T)$ usually a statistical fit is made for extrapolation purposes, using for example some extreme value distribution like Gumbel I. The strength and statistics of the ice loading can be combined to give the design equation by solving for scantlings d from $s(T)$ =r(w,d) (denoting the known relationships as $r(\cdot,\cdot)$, $s(\cdot)$ and $t(\cdot,\cdot)$):

$$d = t(w, T) \tag{12}$$

In order to study the effect of the Limit State, three different limits are investigated (this analysis is following the procedure of Riska and Kämäräinen (2011)). These are given for plating in Table 8.

Table 8. Definition of the limit states for plating.

Limit state (label)	Plating
Elastic (Y)	Stress reaching the yield stress σ_y somewhere in the plate
Plastic (P)	Stress distribution reaching full plasticity somewhere in the plate; permanent deformation still zero
Ultimate (U)	Permanent deformation (w_P) reaching a specified value

The plastic reserve due to membrane stresses is large for plating, and simple formulations for the permanent deformation of patch-loaded plates exist (see, for example, Hayward (2007)). This reference enables determination of the relationship between action and prescribed permanent deflection w_P defined below as w_P/t where t is the plate thickness.

Using the measurements (Figure 7) as a basis, the method shown in Figure 8 can be used to obtain a plot of plate thickness versus return period for different structural limit states (Y, P and three different U's), see Figure 9. This plot can be interpreted in different ways. It is seen that a return period of 100 years (ELIE case) and the FSICR plate thickness gives a structural limit state $w_P/t \approx 0.8$, obtained by interpolation from Figure 9. Alternatively, the design plate thickness and the design ice action (or the corresponding return period in Figure 9) can be used to determine what structural limit is used in each standard. FSICR corresponds to the elastic limit and IACS PC rules to plastic limit while ISO 19906 would allow a large permanent deformation. It should be noted here that the ISO 19906 design value is stated to be derived from Baltic data. The CSR limit state criterion of 70% of yield corresponds to a design ice action of 0.6 MN/m. The corresponding return period is about 2 days, obtained with similar calculation as that provided by Figure 9, i.e. setting the structural limit to $0.7\sigma_Y$ and calculating t_{NET} versus T.

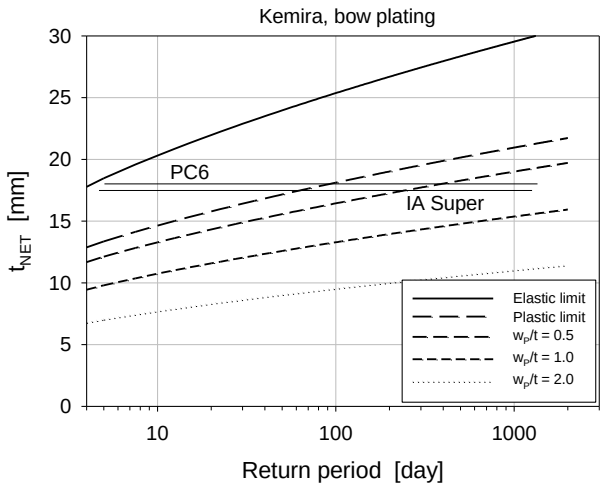

Fig. 9. Plate thickness for the bow area corresponding to different structural limit states plotted versus the return period. Permanent deflection of the plate is w_P and the net plate thickness t_{NET}. The plate thickness according to FSICR and IACS PC rules is also shown (slightly modified from (Riska & Kämäräinen 2011)).

The above discussion shows that the different standards have different logic in defining the Design Point (or Limit State). The logic in each and also the conclusions are shown in Table 9. Three of the standards investigated start by prescribing the design action (FSICR, IACS PC and ISO 19906). This is somewhat in contradiction with the target in ISO to use a given return period for design action, and it can be surmised that the reason for specific formulae in the informative part of ISO19906 is based on the lack of good data from where to derive the needed statistics. All these three standards then also define the structural limit state (even if defining it just 'plastic' is somewhat vague). CSR states only the structural limit state.

Table 9. Summary of the Limit States in the standards investigated.

Standard	Design approach		Return period from measurements
ISO 19906	Design ice action specified	Structural limit state given	$2.7 \cdot 10^6$ years
	5.9 MN/m	'plastic'	
IACS PC	Design ice action specified	Structural limit state given	5.5 years
	2.4 MN/m	'plastic'	
FSICR	Design ice action specified	Structural limit state given	4 days
	0.8 MN/m	'yield limit'	
CSR	Structural limit state prescribed	Design action from measurements	2 days
	$0.7\sigma_Y$	0.6 MN/m	

6. CONCLUSION

Offshore and ship standards have been analysed with respect to the definition of the Limit State given in these standards and in the supporting technical background material. The general analysis has been supported by two practical and relevant case studies; definition of the design temperature and design of (steel) side shell structure. The aim is to find commonalities and differences in these standards.

There is a lot of variation between standards for offshore structures and ships, and the investigation of the Limit States is not aided by the very vague definitions included in these. Some commonality between the different rule sets can be achieved if the Structural Limit given in the standards is interpreted explicitly, and this process was illustrated for (steel) plate structures using Baltic measurements as a basis. The conclusions from this investigation naturally change if a different set of measurements is used, although there do not exist, however, data sets of measurements where loads and responses have been measured for longer than some years. Perhaps the measurements with the Confederation Bridge in Canada can improve this situation.

A reason for performing this investigation was to develop an understanding of the design point or limit state, and to be able to suggest a definition for the 'design point' of local steel structures. Ideally this should include a definition of the allowed structural limit and the frequency of reaching it – if action data matching the frequency would exist. For steel plating a maximum depth of indentation could be used as the structural limit, but for frames setting a structural limit is more difficult because of possible instabilities (frame tripping, web buckling etc.). Further work is needed in these aspects.

The analysis of the design air temperature shows that the different definitions give very different design values. A common industry practice for selection of the design temperature is to use the lowest observed temperature in the operational area. The design temperature definition in the ISO 19906 is closest to this but not the same. This conclusion is likely to change if a different location and data set is used. The emphasis here is that different definitions give widely different values.

The analysis suggests that an explicit Limit State definition would assist the work of a design engineer. Often the Limit State is set by economical or safety considerations which are more general than safety of any individual structure. Society in general accepts the strength and safety level, and perhaps does not accept too low levels of safety. Not knowing where exactly is the lower level of safety leads sometimes to 'design by disaster' where required safety levels or simply required scantling values are increase based on damage or accidents. Conversely, taking an upper level of safety leads to conservative basis and over design situation. This situation is naturally not desirable but shows that much attention should be given to the statistical data set to cover all possible scenarios. In order to facilitate the discussion on the safety levels, it is the author's opinion that in a rational approach all the elements of the Limit State should be stated explicitly and clearly. Another recommendation is that the standards development for floating structures, ships and offshore structures would work towards finding some commonality in the formulation.

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Chapter 17

Session 15: on ISO 19900 WG1 ongoing work

17.1 Presentation & article by Torgeir Moan

NTNU Norwegian University of Science and Technology

International Association of Oil & Gas Producers

Special Session:
Development of the Third Edition of ISO 19900

SC 7/WG 1/TP 1
Limit States & System Effects

Torgeir Moan, NTNU

The 3rd Offshore Structural Reliability Conference
OSRC2016
14-16 September, Stavanger, Norway

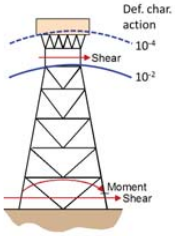
TP 1 Limit States and System Effects Scope & Membership

- Singapore January 2016 WG 1 project
- Technical Panel 1 tasks
  - Define Limit States more clearly – and in a wider sense, e.g. for the stability of floaters (equilibrium LS),
  - Improve guidance on reserve capacity and robustness ; including systems versus component approach
  - Interface towards non-structural modes
- Members – initially appointed
  - Torgeir Moan, NTNU (lead)
  - Sathish Balasubramanian, ExxonMobil
  - Paul Frieze, PAFA Consulting Engineers
  - Kolbjørn Høyland, Dr.techn. Olav Olsen AS
  - Marc Maes, Aleatec AS Inc.
- Members – co-opted
  - Graham Thomas, G. A. N. T. Consulting Ltd

TP 1 Limit States - Activities

- **Review of**
  - **ISO 19900**
    (and the related offshore standards: ISO19902; ISO 19904; ISO 19906)
  - ISO 2394:2015, *General principles on reliability for structures*
  - CPNI (2011). Review of international research on structural robustness and disproportionate collapse, the Centre for the Protection of National Infrastructure (CPNI). *HMSO, 2011*
  - EC1 (2002) Basis of structural design. European Standard EN1990. EU, 2002
  - EC1-1-7 (2006) Actions on structures. Part 1-7: Accidental actions. EN1991-1-7.
  - IMO's MODU-kode av 1989 og IMO' Assembly-resolution 651 (16). International Maritime Organization
- **Proposed modifications of ISO 19900**
- **Prepared an OSRC paper (on USB, web)**

TP 1 Review of ISO 19900



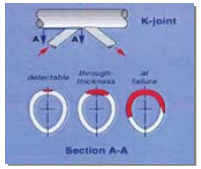
Consistent characteristic actions and action factors

- General
  - **Limit States (LS)** are **physical conditions of the structure** beyond which it no longer fulfills the relevant criteria of serviceability or structural integrity (irreversible damage....)
  - **Limit State Design** is based on LS and an appropriate way to treat **uncertainties** in the predicted behaviour to achieve desired reliability; e.g. by
    - using characteristic values and action and resistance factors or
    - direct measures of reliability or risk
- ISO 19900 refers to
  - Limit States : (SLS), ULS, FLS and ALS
- ISO 2394 refers to SLS and ULS

TP 1 Review of ISO 19900

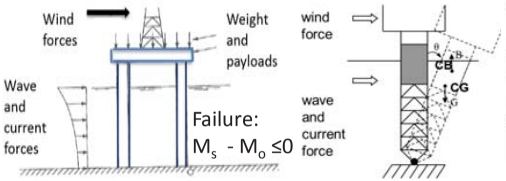
- ULS
  - Maintain both ULS and FLS because of the difference in failure mode and the type of relevant action (Note: in some contexts FLS has been considered SLS)
  - ULS criteria relating to Equilibrium (stability) are not explictly dealt with (neither does ISO 2394). Refer to IMO etc
  - ULS criteria for station-keeping systems need to be elaborated

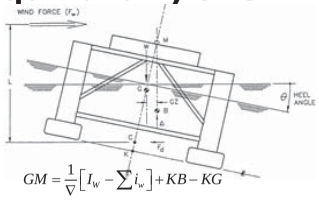
- FLS



- SN data based on alternative definitions of «fatigue faliure»: visible crack; through thickness crack; (member rupture – with different residual fatigue lives and connection to ULS)

TP1 Limit States. Instability (Equilibrium) ULS

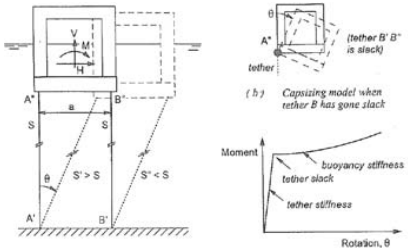


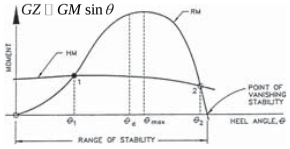


$$GM = \frac{1}{\nabla}\left[I_w - \sum i_w\right] + KB - KG$$

$$GZ \approx GM \sin\theta$$

Stability of «free body» on seabed and articulated tower



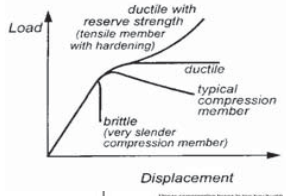


A. Intact stability criteria («ULS»)
-Initial GM
-Area ratio
-Instantaneous GZ
-Intercept limitations (1. and 2.)
B. Damage stability criteria («ALS»)

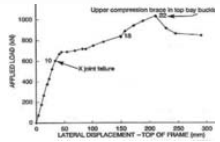
Stability of freely floating platform

Stability of tension-leg platform
(slack in tethers) : Failure: $S_O - S \leq 0$

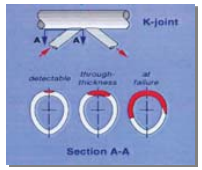
TP 1 Limit States. Strength ULS and FLS



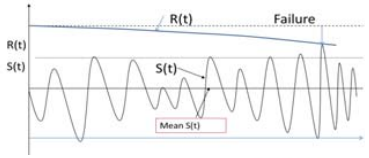
Tubular X-joint (Bomel).



- Ultimate Strength formulations are based on experiments and analysis
  - use strength corresponding to the limit on plastic strain or equivalent cross sectional deformation due to the coupling to high cycle fatigue

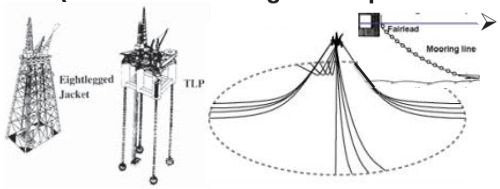


- Interaction between fatigue and ultimate strength

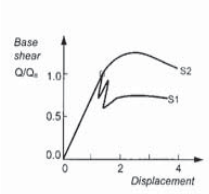


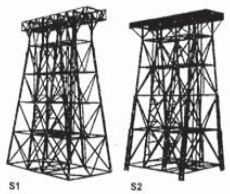
TP 1 Limit States. System Effects

(overload and fatigue component failures)



Jacket Tether - Catenary mooring systems





(with pile groups)

- Systems Limit State
  - global instability
  - sequence of component (overload, fatigue) failure modes
  - feasibility of ultimate strength analysis
- Utilization of ultimate system capacity might imply large local plastic strains or («global» displacements – that might govern the ultimate capacity due to component fracture or exceesive displacements enforced on risers/conductors

TP 1 Review of ISO 19900

- **ALS** is mentioned in **ISO 19900** but not specified
  - seems to refer to local strength check
  - an approach referring to progressive failure initiated by damage should be introduced considering structural damage, flooding/loss of buoyancy events possibly leading to capsizing/sinking or global failure (referring to robustness in a more general sense)
- **ISO 19904** refers to IMO codes for stability:
  Damage stability is based on a two-step procedure:
  - estimate or use specified damage condition
  - verify stability for damaged structure w.r.t. capsizing or progressive flooding

TP 1 Review of ISO 19900
System Effects: Robustness

- The current version of ISO19900 (2013) defines **robustness** by
  "the ability of a structure to withstand accidental and abnormal events without being damaged to an extent disproportionate to the original cause",

- Other motherhood codes for structures: ISO 22111 (2007), ISO 2394 (2015), EC1(2002) and ISO 19900 (2013) also mention that:
  - Robustness or damage (fault) tolerance are desirable features of structures
  - *these codes replace accidental and abnormal events by: fire, explosions, impact or the consequences of human error*

  - Even if the notion of robustness is not explicitly used in connection with buoyancy and (rigid body) stability of floating structures, it is also a desirable feature in this connection

TP 1 Review of ISO 19900
Robustness

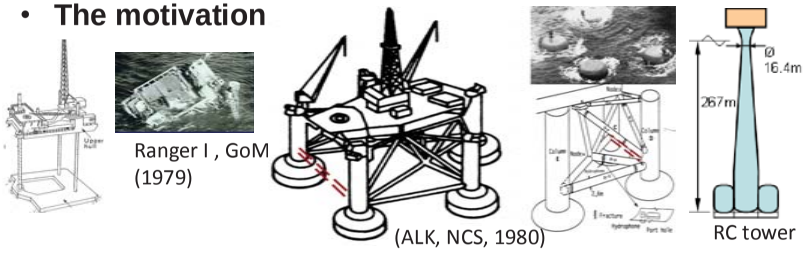
- The definition of robustness implies **damage tolerance**; i.e. that the damage should not escalate or progress into more serious consequences.

- Damage tolerance is also a crucial property for deteriorating structures to ensure the necessary safety when relying upon monitoring or inspection to detect cracks or corrosion damage etc. and initiate the necessary remedial actions in case damage is detected. The cited definitions of robustness do no reflect this feature (recognised early in aviation,…)

- On this basis the following definition is suggested for consideration:
  "the ability of a structure to limit the escalation of accident scenarios (floating positions; or structural damages)- caused by accidental actions and abnormal strength due to fabrication or deterioration phenomena - into accidental conditions with a magnitude disproportionate to the original cause"

TP Review of ISO 19900
Make Structures Robust by Design

- **The motivation**



Ranger I , GoM (1979)

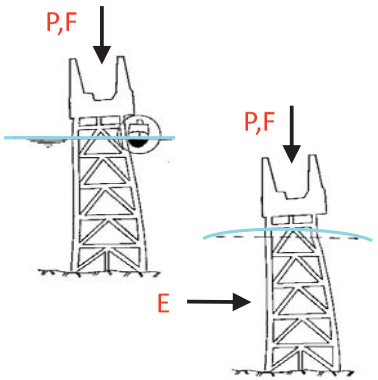
(ALK, NCS, 1980)

RC tower

- **Design by either of the following approaches**
  - designing the structure locally to sustain accidental actions and other relevant simultaneously occurring actions (***key element design***); **ULS -**
  - designing the structure by accepting local damage but require the damaged structure to survive relevant actions (***alternate path design***). **ALS - type criteria**

  The latter approach is not operationalized in ISO19900

TP 1 Accidental collapse limit state (for structural integrity) - ALS, also denoted PLS – local strength or system check



Motivation

- experiences with accidents

Example Procedure (NPD/Norsok):

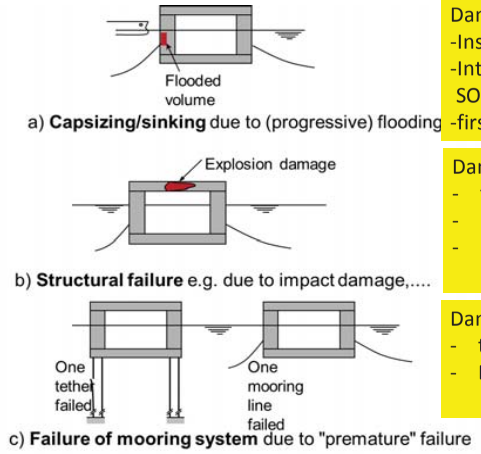
Step 1

- check capacity to resist abnormal or accidental loads with annual exceedance probability of 10^{-4} (allowed to cause local damage only)

Step 2

- check that the structure in damaged condition (**step1** or **specified damage**) does nor experience total collapse for actions with annual exceedance probability of 10^{-2} or 10^{-1}(when initial damage is not correlated with the metocean actions)

TP 1 Accidental collapse limit state in general - Example: the Norsok approach



Damage stability
-Inspired by Titanic accident in 1911
-Introduced for ships through SOLAS in 1948
-first codes for MODUs in 1970s

Damage tolerance for
- the hull by NPD 1984
- (after Alexander Kielland,
- but inspired by Ronan Point building collapse

Damage tolerance for
- the mooring
- DP systems

TP 1 Review of ISO 19900

"Accidental collapse limit state" in view of deterioration (fatigue, corrosion)

- High cycle fatigue design criterion should depend on consequences and inspection plan and quality
- Consequences may be linked to a robustness (ALS) criterion
- Low cycle structural and soil degradation in ultimate strength assessment (depending on environmental conditions, seismicity)

TP 1 Review of ISO 19900

- **Classification of uncertainties**
  - normal – fundamental variability, statistical, model uncertainties of natural and *man-made* actions and resistances
  - gross errors – resulting in accidental actions or deficient strength

Uncertainties in resistances versus action effects

- action effects are specified by analysis procedures of various refinements

Example:
Wave actions/effects/: regular wave (ULS), irregular short-term/longtrem wave conditions
Morison's formula; potential theory (linear/nonlinear, CFD), model tests – implying different uncertainties

- versus uncertainties accounted for in safety factors or margins in design criteria

- Choice of consistent characteristic actions and action factors

TP 1 Contents List for the Main Text/Annex

1. **Introduction**
   - General background – based on ISO 2394, IMO and the discrepancies between these standards and ISO 19900

2. **Limit states**
   - **Highlight principles for establishing limit states**; i.e. Limit States should preferably
     - be based on the first principles of mechanics theoretical formulations (not pure regression to data)
     - explicitly contain the parameters of influence
     - have as a small (random) model uncertainty as possible
     - be as simple as possible
     - be feasible to analyse.
   - Define more clearly **Stability (Equilibrium) Limit states** (ULS, ALS); especially for floating platforms
   - Elaborate on **Limit states** (ULS, ALS) for station-keeping systems
   - Definitions of ULS and FLS as well as coupling between ULS and FLS for structures and foundations (soil)
   - ALS approaches
     - local strength versus two step system approach

TP 1 Contents List for Annex

3. **Uncertainty measures versus action effect and resistance factors (margins)**
   - Uncertainties in actions and action effects
     - principles for nonlinear global structural/soil systems analysis
   - Uncertainties in resistances
4. **Robustness**
   - Operationalize the ALS (PLS) design check to ensure robustness ; i.e. - as a local strength check or as a two-step system design check for structural and stability modes
   - Methods of structural ULS/ALS analysis (nonlinear/linear global)

Miscellaneous issues

- Classification of actions (based on origin – function, environment, accidental; based on probability , by exceedance probability, and avoid abnormal/rare
- Action combination, design conditions
- Harmonization of terminology in the ISO 19900 series of standards

TP 1 Impact on Normative Requirements

- ISO 19900 should deal with general principles, limit states, methods for assessing uncertainties, reliability and risk, verifying compliance with relevant acceptance criteria

  While ULS, FLS criteria will be generally applied,

  ALS criteria will be applied depending on the safety (consequence) class.

  The definition of characteristic values, safety factors and

  - margins could be defined in Regional annexes.

  Different target safety levels can then be achieved depending on the statutory regime
- Improved consistency of principles, limit states, terminology for the ISO 19900 series standards could then be achieved

The 3rd Offshore Structural Reliability Conference
OSRC2016
14-16 September, Stavanger, Norway

Limit States and Systems Effects of Offshore Structures with Emphasis on Design for Robustness

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Email: torgeir.moan@ntnu.no

ABSTRACT: Modern design codes are based on Limit State Design; which involves defined limit states and an appropriate approach to treat uncertainties in the predicted behaviour to achieve the desired reliability. The focus herein is to define limit states, i.e. physical conditions of the structure beyond which it no longer fulfils the relevant criteria of serviceability or structural integrity (e.g. irreversible damage...) for individual components and the system, considering ultimate and fatigue failure modes. Both equilibrium (e.g. global stability) and strength modes are considered. In particular the use of Accidental Collapse Limit States to ensure robustness – or damage tolerance – in view of damages caused by accidental actions and structural deterioration, is dealt with. It is found to be important to define e.g. the ultimate and fatigue limit states so that the effect of fatigue and other deterioration on the ultimate strength is accounted for. It is suggested to harmonize stability and strength requirements for floating structures.

KEY WORDS: Limit States for Equilibrium (Stability) and Strength, ALS design check against progressive failure initiated by accidental events. Damage tolerance for use of inspection and repair. Robustness.

1 INTRODUCTION

Modern designed codes for marine structures are based on defined criteria – or – limit states for serviceability and safety expressed on a capacity versus demand format [1-3]. Normal uncertainties due to fundamental variability or limited data or model uncertainty in those quantities are accounted for by semi-probabilistic design using characteristic values of the variables and adequate partial safety factors to ensure an acceptable safety level. ULS criteria are typically specified by metocean actions with an annual exceedance probability of $10^{-2}$and action factors of the order of 1.2-1.4. Alternatively a lower exceedance probability and action factors might be used when the target reliability is of the order of 10^{-4}. The latter approach might give more consistent safety criterion. Alternatively, a direct probabilistic approach in which the variability and uncertainty of the variables are used to obtain a measure of the notional failure probability which serves as a basis to ensure a design which complies with the desired target failure probability.

Human actions which influence design, fabrication and operation of structures, and, hence, man-made loads and the resistance of fabricated structures, normally follow defined procedures, which ensure that the corresponding uncertainties in structural resistances, loads an load effects follow certain patterns, which make it possible to measure and predict such uncertainties, which are denoted normal uncertainties. However, sometimes the human actions depart from intended practice and result in so-called human errors, or gross errors; e.g. [4-6].

The distinction between normal uncertainties and gross (human) errors is useful in deciding how the associated risk shall be controlled. To ensure that the likelihood of failure due to normal uncertainties is within acceptable limits, factored characteristic load effects and resistances are applied in the design equations as mentioned above.

Clearly, the nature of human errors differs from that of natural phenomena and "normal" man-made variability and uncertainty, and different safety measures are required to control error-induced risks. Primarily, gross errors and their effects should be avoided by adequate competence, skills, attitude and self-checking of those who do the design, fabrication or operation in the first place; and by exercising "self-checking" of their work. In addition, quality assurance and control (QA/QC) should be implemented in all stages of design, fabrication and operation. This implies for instance control of the analyses and design as well as inspection/monitoring and repair of the structure during fabrication and operation. This effort also involves inspection/monitoring relating to process equipment, ballast system, watertight doors and pipes and the operation if such facilities, in order to control e.g. possible hydrocarbon leaks that can result in fires and explosions; ballast faults. In particular, a follow-up of deterioration due to crack development and corrosion is needed during operation. In addition, some accidental scenarios develop over time and they can be controlled to some extent (Event Control). For instance, fires and explosions may be controlled by detecting leakage of combustibles (hydrocarbons) and activating shut-down, fire extinguishing, etc. Finally, the risk especially associated with fabrication and operational human errors can be controlled by introducing a progressive or accidental limit state for the system, as discussed subsequently. In this paper only the principle of Limit State Design and especially Limit States are addressed.

Limit States (LS) are physical conditions of the structure beyond which it no longer fulfils the relevant criteria of serviceability or structural integrity (irreversible damage....).

Limit State Design is based on LS and an appropriate way to treat uncertainties in the predicted behaviour to achieve desired reliability; e.g. by - using characteristic values and action and resistance factors or direct measures of reliability or risk.

ISO 19900 refers to the following Limit States: SLS, ULS, FLS and ALS but does not explicitly deal with the ULS, ALS stability of floating structures (neither does ISO 2394 [1]); and to a limited extent with station-keeping systems. It is noted that the current ISO 19900 set of LS deviates from ISO 2394 and EN Basis of Design, which only refer to SLS (serviceability) and ULS (safety) Limit States. ISO 19906 [7] refers to ALS denoted abnormal LS. Moreover, this standard refers to characteristic values with probabilities in the range 10^{-2} to 10^{-5} for ULS and ALS. Moreover, the ISO standards [8-10] should be checked with respect to harmonization.

As indicated in Table 1, normal uncertainties may then be accounted for in serviceability, ultimate and fatigue limit states for components, while gross errors are considered in connection with accidental or progressive limit state requirements to the system. In this way there will be no conflict in considering natural randomness and "normal" uncertainties for the usual component ultimate and fatigue limit states. Quantitative measures of safety to deal with the different limit states, are obtained by using classical structural reliability analysis (for SLS, ULS and FLS) and by using risk analysis (for ALS or PLS).

Table 1 Causes of structural failures and risk reduction measures, see, e.g. [6]

Cause	Risk Reduction Measure
Less than adequate safety margin to cover "normal" uncertainties in actions and resistances	Increased safety margins in (SLS), ULS, FLS; or inspection (FLS)
Gross errors in design fabrication operation	Design QA/QC Event control Inspection/repair Design for damage tolerance (ALS or PLS)
Unknown phenomena	R&D and implementation of the findings

Design criteria may be split into serviceability (SLS) and ultimate limit state (ULS) criteria. The latter category in general comprises criteria for structural integrity, and may further be classified in terms of (ISO 19900 [2]):

- Ultimate limit state of components
- Fatigue limit state of components

The ULS criteria are split in two classes; namely loss of equilibrium (instability) and strength (ductile failure, fracture or excessive deformation) criteria. As mentioned above, safety criteria are expressed by capacity and demand. Stability criteria are expressed in terms of external forces/loads both for the capacity and demand; i.e. the capacity is expressed by (stabilizing) loads. In principle both low cycle and high cycle fatigue needs to be considered. Moreover, as discussed subsequently, a low cycle damage analysis check needs to be carried out when the full ultimate capacity in a pushover mode is utilized.

In view of human errors the structural engineering community has in several decades been concerned with providing guidance to make the structures fail-safe or robust against such events, especially when human lives are at stake.

It is interesting in this connection to note that damage stability criteria, which are ALS-type criteria, were discussed in decades, catalysed by the Titanic accident on 20. January 1914, and were introduced first qualitatively, later quantitatively for sinking/ instability of ships in the 1948 SOLAS Convention [11]. A reason for the late acceptance of damage stability criteria was the complexity of the analysis to demonstrate compliance. Damage stability criteria were introduced in the early mobile platform rules (e.g., [12, 13]). The damage stability check has typically been specified with a damage specified to be one or two compartments flooded – relating to ship impacts. The ALS criterion was formally introduced for all failure modes of offshore structures in Norway in 1984 [5, 14, 15]. While previous accident experiences justified this approach, the Alexander Kielland accident made it possible [5,16]. Contrary to the UK building codes and damage stability requirements, the NPD requirements were more functional based on damage scenarios that had to be assessed by risk analysis. For instance, such analyses were applied to demonstrate that the conventional compartmentation of floating steel structures was not necessary for the Heidrun TLP concrete hull - in view of ship impacts, dropped objects and ballast faults [17]. Another aspect that it was found necessary to include consideration of damage due to abnormal environmental loads as well as "damages" implied by fabrication errors and the potential influence of human errors on fatigue and other deterioration phenomena.

Risk acceptance differs between jurisdictions and consequence class. For instance, it is noted that ALS criteria do not apply if there is no risk of fatalities, severe environmental damage or significant economic loss potential [14]. However, the principles of limit states and robustness are a common feature of all risk assessment processes that in all cases they must form a decision-making framework that leads directly to a decision about the design approach deemed appropriate or necessary.

The safety philosophy emerging in modern design codes is based on the following general safety principles:

- Structural integrity to withstand environmental and operational loading throughout its lifecycle.
- Prevent occurrence of and protect against accidental events
- Tolerate at least one failure or operational error without resulting in a major hazard or damage to structure.
- Provide measures to detect control and mitigate hazards at an early time to avoid accident escalation.

The focus herein is on the limit states in a broad sense. The mathematical expressions for the limit states will be used as a basis for:

- Failure function for reliability analysis. In this case the parameters in the limit state are considered to be random variables.
- Design equations. In this case the parameters in the limit state – according to the partial factor method – are given by their characteristic values, each multiplied by a partial safety factor.

Limit states should preferably

- be based on structural mechanics theoretical formulations (not pure regression to data)
- explicitly contain the parameters of influence
- have as small (random) model uncertainty as possible
- be as simple as possible
- feasible to analyse.

To estimate damage, i.e. permanent deformation, rupture, etc., of parts of the structure, nonlinear material and geometrical structural behaviour need to be accounted for. While in general nonlinear finite element methods need to be applied, simplified methods, e.g. based on plastic mechanisms and simplified fracture criteria, can be developed and calibrated using more refined methods, to limit the computational effort required in practice.

2 ULTIMATE EQUILIBRIUM LIMIT STATE (UELS)

2.1 General

The equilibrium limit states considered herein comprise:

- overturning of structures resting freely on the seabed
- articulated tower
- loss of tension in pre-tensioned structures
- global instability (capsizing) of freely floating structures

2.2 Overturning of structures resting freely on the seabed

Figure 1 shows a structure with a free support on the seafloor, under various loads, some of which tend to overturn the structure and some loads tend to resist this overturning. In this case, the ultimate limit state can be expressed by the moments relative to overturning point O. If the corresponding moments are denoted M_o for load type *i* and M_o for load type *j*, respectively, the criterion for instability can be written as:

$$M_s - M_o \leq 0 \qquad (1)$$

where the moments M_o and M_s generally are calculated by the forces causing clockwise and counter clockwise moment in Figure 1. It should be noted that in a semi-probabilistic design format the load factors on the unfavourable moment, M_o and the favourable moment, M_s are larger and less than 1.0, respectively.

The moment (M_o) is determined based on a high characteristic environmental loads relating to ULS check. When considering the stabilizing moment due to (net) structural weight and deck live loads, a low characteristic value of the variable loads is applied.

A particular issue relating to the present failure mode is whether the check is done with quasi-static forces/loads or a dynamic check. The effect of dynamics would primarily be inertia forces which will have a stabilizing effect.

If the structure is provided with an embedded foundation, the design check will be replaced by limit states relating to the failure modes of the foundation and soil.

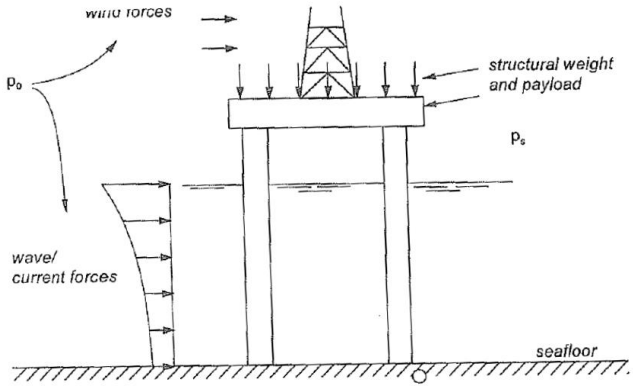


Figure 1. Scenario for overturning failure mode for a structure resting freely on the seabed.

2.3 Stability of an articulated tower

Figure 2 shows an articulated tower subjected to an overturning moment due to waves, current and wind as well as gravity when inclined. This overturning moment is resisted by buoyancy. For this system the limit would be the static and/or dynamic angle of heeling relative to possible deck airgap, downflooding, etc.

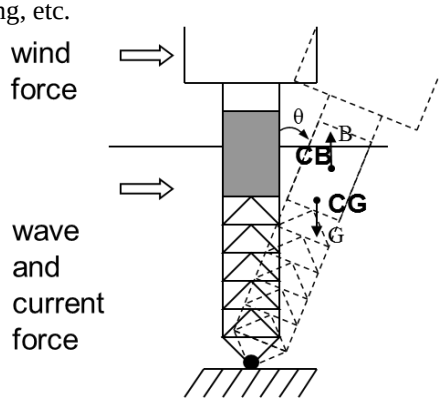


Figure 2. Articulated tower.

2.4 Loss of tension in pretensioned structures

The tension-leg platform is kept in position by excessive buoyancy and a pre-tensioned mooring system. When the structure is subjected to the resulting forces, H, V and M (Figure 3a), due to waves, current and wind, it undergoes a horizontal displacement and thereby a vertical "set-down" (due to the angle θ). The resulting tension in the tethers will partly be larger, and partly smaller than the pretension.

Commonly, tether slack (and not the eventual "capsizing") is considered the ultimate limit state. In this connection distinction may be made between "slack" in two tension-legs (Figure 3) and one tension-leg (in case of a diagonal wave condition). The former condition may be considered more critical than the latter one. "Slack" may be considered a limit state partly because the M-θ diagram (Figure 3) exhibits a "limit" when slackening occurs, but also because going "slack" may imply large bending moments in the tethers, possible malfunction of end supports or a snatch load when it goes into tension again, after being "slack". Moreover, the

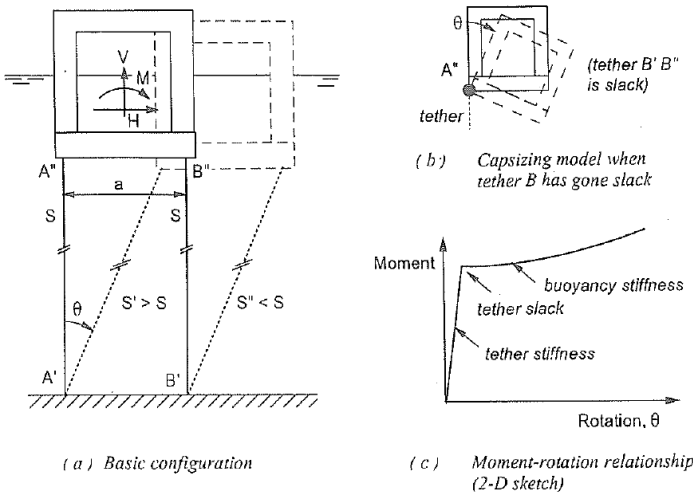
prediction of this kind of behaviour after "slack" is a much more complex task than just predicting the onset of "slack". Tethers, hence, the ultimate limit state (relating to slack and hence "instability"):

$$S_0 - S \leq 0 \qquad (2)$$

where S_0 and S are the axial pretension and axial ("compression") force due to environmental load, respectively.

The criterion (2) may be refined by splitting the effect of environmental loads into tension due to: mean offset, low-frequency, high-frequency and wave frequency load. It should be noted that a slack in a tether would imply slack in the other adjacent tethers in a platform corner and normally those in another corner as well. In addition, the tether needs to fulfil structural and foundation strength criteria

It should be noted that the slack condition, especially in irregular wave conditions has a short duration. This feature would imply large inertia forces on the hull, and limit the displacement of the tether attachment. Hence instead of conservatively using slack as a criterion the actual stress level in the tethers may be used in conjunction with the relevant structural limit states (normally with reference to first yield).



(a) Basic configuration

(b) Capsizing model when tether B has gone slack

(c) Moment-rotation relationship (2-D sketch)

Figure 3. Scenario for tether slack in TLPs.

2.5 *Capsizing of freely floating structures*

Current stability criteria for catenary moored floating platforms and ships primarily account for the effect of wind, while wave and current forces are not explicitly taken into account (e.g. [18,19]). The stability criteria essentially refer to heeling which may cause flooding and eventually capsizing, see Figure 4. The relevant parameters are then the moment acting on the floating structure and the corresponding angle of heel. Most regulatory authorities (e.g. The International Maritime Organisation, IMO [20, 21]) and classification societies (e.g. ABS, DNVGL and LR) have stability standards for vessels and semi-submersibles operating under their jurisdiction.

The stability criteria for floating platforms are commonly expressed in terms of: the criteria listed in Table 2.

Table 2 Stability criteria for floating platforms

A. Intact stability criteria («ULS»)

Initial GM
the relative magnitude of the areas under the curves for M_H (area B+C) and
M_R (area A+B). See Figure 5a.
Instantaneous GZ
the maximum static angle (θ_1) and maximum angle of heel (θ_2) at which
"capsizing" occurs (for $\theta > \theta_2$, the overturning moment is greater than the
uprighting moment). (Figure 4)

B. Damage stability criteria («ALS»)

- Maximum first intercept
- Minimum residual GZ
- Minimum range of stability
  for specified damage condition
  the relative magnitude of the areas under the curves for M_H and M_R. See Figure 5.b

The parameters GM and GZ are defined by

$$GM = \frac{1}{\nabla}[I_w - \sum i_w] + KB - KG \qquad (3a)$$

$$GZ \approx GM\sin\theta \qquad (3b)$$

where K - keel centre; G - centre of gravity; B - centre of buoyancy; F_w- wind force; W - vessel weight; C - centre of underwater resistance; L - wind heeling arm; F_d - drag force (=F_w); GZ - vessel righting arm. KG -height of centre of gravity above keel; Δ-vessel buoyancy (=W); Heeling moment: HM=Fx•L; Righting moment: RM=W•GZ; θ_1- first intercept and steady heel; θ_2 - second intercept; capsize; θ_d - downflooding angle and max = angle at max. Righting arm. It is noted that the first and second term in GM dominate for semisubmersibles and spars, respectively. Typical GZ curves are given e.g. by Clauss et al. [18].

The (static) heeling (overturning) moment caused by wind is typically calculated by:

$$M_H = \int F \cdot a \cdot dA \qquad (4a)$$

where F is the wind drag-force and a is the moment arm, which is taken to be the distance from the centre of the force to the centre of hydrodynamic forces which are assumed to balance the driving wind forces. This is clearly an approximation when the platform is kept in position by a mooring system or thrusters. For new structures for which there is no previous experience with wind forces, experimental verification of M_H is required.

The stabilizing (righting) moment, M_R is expressed by

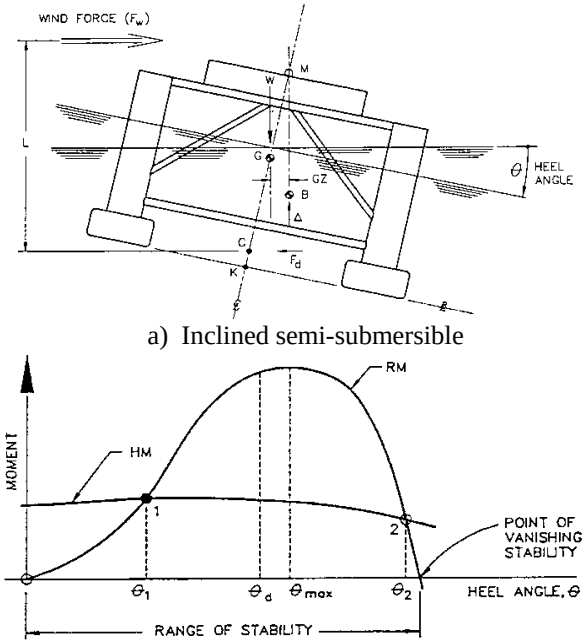
$$M_R = \Delta \bullet GZ \qquad (4b)$$

where Δ and GZ are the displacement and "righting arm", respectively.

The initial metacentric height, GM_0 is also an important "stability" parameter. If GM_0 is negative, the vessel is unstable. However, commonly the centre of buoyancy shifts as the heeling angle increases, resulting in an additional stabilizing moment, and the structure will attain a new position of equilibrium when the angle of heel become large.

So far, static heeling moments due to wind and righting moments due to hydrostatic (reaction) forces, have been considered. Simplified dynamic stability considerations are used to be considering the energy associated with M_H and M_R, as illustrated in Figure5.

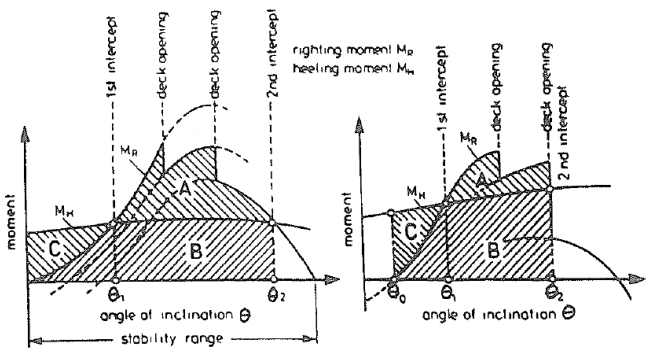
Clearly, since only steady wind forces are considered to contribute to the heeling of the floating structure, the approach will be simplified in view of the existence of wave-induced motions. The current "energy-based" criteria (discussed below) are therefore calibrated by model tests and in-service experiences.



a) Inclined semi-submersible

b) Heeling and righting moment versus angle of heel

Figure 4. Terminology related to stability of freely floating structures.



a) Intact stability b) Damage stability

Figure 5. Illustration of moment versus heel angle curves for intact (a) and damage stability (b) criteria for floating (semisubmersible) platforms [18].

The area criteria utilize the energy relating to overturning and stabilizing forces.

Different criteria are applied for different types of floating platforms used as mobile drilling units. The main reasons to differentiate criteria are: different uncertainties in calculating M_H and M_R as well as the consequences of failure.

The wind speed should, in principle depend on the local conditions, and, e.g., correspond to a given return period specified for the type of structure. However, for exploratory drilling units, fit for operations world-wide, the following velocities are commonly used: operation (70 knots); "survival" (100 knots) and damage condition (50 knots).

The criteria for damage stability are indicated in Table 2. In addition the magnitude of damage needs to be specified. For mobile units flooding of one or two compartments adjacent to the sea are typically considered. These damage conditions are based on operational experiences with collision damage on steel hulls. A concrete hull will have more resistance to ship impacts and the corresponding damage conditions need to be reconsidered. Accident experiences for offshore structures are recorded in WOAD [22]. Incident and accidents for semisubmersibles in the period 1974-1998 are listed and discussed by BMT [19]. They show that also the risk of dropped objects and ballast errors need to be considered. The Ocean Ranger accident in 1984 directed the attention to buoyancy loss, e.g., due to ballast error, which in principle could occur in any compartment with variable ballast. This would imply consideration of flooding of any compartment with variable ballast.

The Alexander Kielland accident leads a requirement of reserve stability, without referring to which damage should be tolerated. This requirement normally implies account of the buoyancy of the deck structure; and, hence, implications on limit states relating to ultimate strength of the deck structure.

It is necessary to carefully examine whether these damage conditions apply for unique structures -such as floating production structures (Figure 6). This situation suggests that the damage conditions should be assessed by risk analysis.

P-34 in Brazil Thunder horse in GoM

Figure 6. Loss of buoyancy events for floating production platforms

Semi-submersible mobile units have been lost on a number of occasions after damage and flooding, notably the Alexander Kielland in 1980 [16] and Ocean Ranger in 1982 [23, 24] accidents, mostly after major structural damage has occurred or serious errors in ballast control have been made by the crew. Deficiencies in the stability criteria as such do not seem to be a factor in these accidents.

Established intact stability standards have developed as a result of a long historical process, and seem to have been

completely successful in avoiding capsizing of semi-submersibles when in the intact condition. This success has led certain investigators to ask whether established stability standards may in fact be too conservative, and may perhaps lead to unnecessary costs. [25, 26] noted that the conventional area ratio criterion provide a certain level of conservatism. Several authors have criticised the fact that the first down-flooding angle, used in an intact stability analysis, is normally defined relative to a still water surface. The angle at which openings enter the water will in practice depend on the motions of the vessel and of the water surface. ABS [27] subsequently developed stability criteria which were intended to be more closely related to the physical phenomena; i.e. the motions. The new ABS intact stability criteria eventually adopted in paragraph 4.6.5 of the IMO Code on Intact Stability [20], for a restricted type of units.; as an 'example' of alternative intact stability criteria.

3 STATIONKEEPING SYSTEMS

3.1 General

The basic systems for station-keeping are:

- mooring system, consisting of structural components such as cable/chain, anchors, fairleads, ...
- thruster system, consisting of propellers, power transmission, power supply, control systems, ...
- tension-leg system, consisting of vertical tubular members

The systems are made up of a certain number of the basic components, which are arranged in different layouts. While the former two systems keep the structure in position in the horizontal plane, the tension-leg system maintains station in all degrees of freedom. A failure of a tension-leg system is hence more critical than failure of the former two systems.

The spread mooring system has been the most commonly used mooring system up to now. The turret mooring system maintains the horizontal position and orientation of the turret by a special mooring system. The structure can rotate freely around the turret.

The use of thruster systems for drilling, accommodation or production systems under severe weather conditions has primarily been to assist mooring systems. But pure dynamic positioning systems have been used for other services and under less severe weather conditions than e.g. in the North Sea.

When the thrust system is intact and available, the thrust force may be regulated between zero and some maximum value, which can be determined theoretically or experimentally. However, failure of components (power, transmission, control) may lead to failure of the thruster force in one or more thrusters.

The direction of the force is controlled by a steering system, which should be available and correctly operated. Reliable operation of the steering system is obviously decisive, because erroneous operation of this system can even make the thrust add to the loading on the mooring system, rather than the opposite. Usually, it is distinguished between manual and automatic steering system.

3.2 Ultimate Limit State

Limit states for station keeping systems include

- serviceability criteria related to offset (which affects loads on e.g. risers)
- safety criteria are associated with
  - failure of one or more components of the station-keeping system
  - excessive motions
  - drift-off (e.g. due to progressive failure of the mooring system)
  - mooring-induced capsizing (tripping)

which are listed in increasing order of importance.

Failure of individual components in a catenary mooring or DP system rarely would represent serious consequences. It is the excessive motions or drift-off that represents the largest risk, as they, for instance may lead to:

- collision between the actual rig and other installations
- impacts or wear on pipelines or other subsea equipment, caused by dragging anchors;
- blowout, fire or explosion or oil spill;
- possible grounding:
- failure of risers or bridges, depending upon the activity that takes place

Obviously, such consequences would depend on the operation, the prevailing weather at the time, the number and location of other installations, whether the installations are manned or not, the availability/reliability of possible active actions to control the motions or drift-off, etc.

Ultimate limit states associated with the strength (and flexibility) of the mooring lines (and other components) are defined to avoid the failure modes indicated above. The strength limit states of mooring lines have normally been expressed by the tension, in the following way

$$\mathrm{R} - (T_0 + T(Q)) \leq 0 \qquad (5)$$

where T_0 and $T(Q)$ are the pretension and the tension due to environmental actions, respectively. R is the resistance. It should be observed that local bending stresses may occur, especially in chain links e.g. at fairleads. Some design standards indicate that such stresses should be considered in the design check.

The ULS for thrusters in a DP system is based on an analogous capacity and demand formulation where the capacity is expressed by the propeller thrust.

3.3 Fatigue Limit State

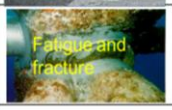
Moreover, fatigue criteria in terms of cumulative damage based on SN-curves need to be checked. While often SN curves in terms of tension has been used in FLS checks, possible local bending effects should also be considered.
However, it should be noted that possible local bending effects at the fairlead and anchor, etc., are included in the limit state.

4 LIMIT STATES FOR STRUCTURES AND FOUNDATIONS

4.1 General

Relevant limit states for structures are listed in Table 3.

Table 3 Limit states for structural strength

Limit states	Failure mode	Remarks
Ultimate (ULS) -Overall stability - Ultimate strength of structure, mooring or possible foundation		Different for bottom – supported, and buoyant structures. Component design check
Fatigue (FLS) - Failure of welded joints due to repetitive loads		Component design check depending on residual system strength and access for inspection
Accidental (ALS) - Ultimate capacity[1)] of damaged system with "credible" damage		Operationalized robustness design check of the structural system

Limit states are subject to a systematic and random uncertainty. As discussed below, the systematic component should be as far as possible be accounted for directly, while the random component implies a partial safety (material) factor.

Simplicity in the design formulation implies a reduced chance of gross errors.

It is noted that corrosion normally is not a limit state but have an effect on the capacity. However, in connection with follow up of structures in operation where a corrosion allowance has been assumed, a limit state in terms of residual corrosion allowance might be formulated.

For the ultimate strength limit states considered the capacity is obviously expressed by stress or force. However, since there might be inherent cracks, the ultimate strength determined by numerical analysis, should in principle by a certain plastic strain level or in a more simplified manner by "equivalent" deformation limits. In experiments the most convenient would be observation of cracks or equivalent (plastic) deformation.

4.2 Ultimate strength of structures

4.2.1 *Structural Steel Components*

Marine metal structures are made of beams, stiffened panels, shells, and joints. Limit states should, as far as possible, be based on strength of materials, elastic buckling formulae, plastic limit loads, etc., and appropriately modified to account for residual welding stresses and geometrical imperfections, and should not be obtained by a pure regression analysis of test results. It would also be advantageous that a formulation converges to, say, elastic buckling formulae and yield stress for very slender and stocky components, respectively.

Figure 7a indicates various load-deformation (displacement, rotation, ovalisation) characteristics for metal components. The behaviour depends on the material characteristics (stress-strain relationship), the geometry and type of loading.

A member with ideal elasto-plastic behaviour and no significant defects, would exhibit a clear ultimate limit (illustrated by the ductile curve). A tensile member with strain hardening has got an increased capacity. The tensile capacity of a member made of base material may be expressed by the net cross section area and the stress: $(\sigma_y+\sigma_u)/2$. However if it is a welded member there will be initial defects, which might increase in size during operation. While for instance mild steel might have a strain of 20-30 % at failure, the presence of cracks will reduce this limit [28]. In nonlinear numerical analysis of the strength this fact may be accounted for by introducing plastic strain failure criterion. This becomes particularly important where high stress concentration might occur. Hence, in connection with the definition of ultimate strength of tubular joints, the capacity both when it is based on experiments or numerical analyses will be determined by assuming a plastic strain limit. In practice the strain criteria can be transformed into a deformation criterion. For instance joints with brace(s) subjected to axial forces the deformation limit in terms of a 0.06 D ovalisation of the cross section has been used.

Figure7b shows load deformation curve for an x-brace system where ovalization of the tensile member is caused by the compression member in the tubular x-joint. If this action level is exceeded under cyclic actions, failure will occur. For members in axial compression buckling will limit the capacity, and fracture etc. is not relevant and the component will exhibit a "brittle behaviour"- see Figure 7a.

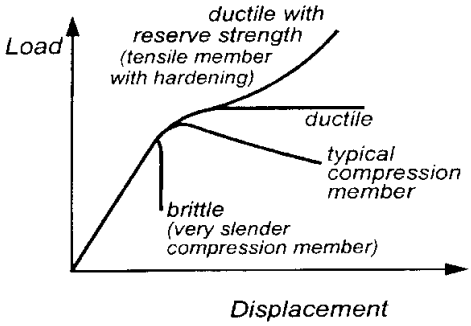
Ultimate limit states may be expressed by parametric formula, numerical calculation procedures (e.g., nonlinear finite elements), or by testing. Existing component ultimate limit states are generally given by parametric formulae in terms of stresses, forces or moments; and exceptionally by strains, which have been justified by test results. Examples are shown by e.g. [29].

There are significant differences between formulae used in different codes for the same failure mode. Hence, there is a need to compare and harmonize strength expressions, to identify the "best" one. The capacity is governed by "local" failure due to buckling collapse of parts of the components.

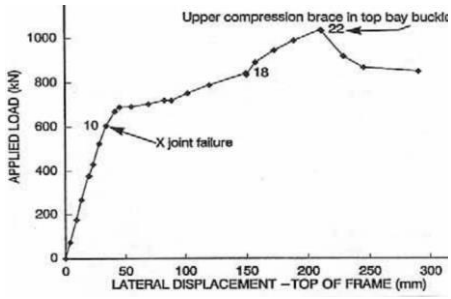
A particular issue of concern in connection with buoyant marine structures is the definition of ultimate limit states in view of conditions where large deformations/strains cause local failure/fracture and, hence, possible leakage. "Water tightness" may in principle be considered as a separate limit state, which may imply a lower limit on deformations than dictated by the pure load carrying capacity. Generally, several formulations are applied for different limit states. Significant efforts would be required to compare existing formulations to establish the "best" approach.

The current availability of accurate numerical methods allows determination of the strength on a case basis as well as to accomplish systematic studies of complex structures. A particular advantage of numerical methods is that the effect of each factor can be studied separately. However, numerical methods to trace nonlinear behaviour up to collapse are susceptible to various sources of uncertainty, including human errors of different kinds. It is, therefore, necessary to standardize the relevant numerical analysis procedures. It may even be necessary to "certify users" and validate results before such analyses can be used by designers at large to determine

resistances. Up to now, ultimate strength formulations in design codes have been established primarily on the basis of simple structural mechanics models, with appropriate "knock down" factors determined experimentally. Relatively simple and accurate formulations are achievable when failure mode follow some simple mechanism (buckling, yield hinge mechanism, etc.). To achieve such simple failure modes, e.g., for complex stiffened, thin walled plates and shells, the stiffeners are considered to meet certain assumptions, which commonly are conservative.



a) Typical load deformation relations



b) Load-deformation for tubular X-brace [30]

Figure 7. Metal component behaviour for definition of ultimate limit states.

4.2.2 *Structural Concrete Components*

Offshore concrete structures are built up by plates, cylindrical and spherical shells as well as joints between such components. Their capacity is generally described by the axial, bending and shear capacity. For shell structures subjected to external pressure, buckling is also of relevance and the strength is referred to as the implosion capacity.

4.2.3 *Foundations (soil-structure)*

Limit states need to be defined for various types of foundations such piles, buckets, gravity type foundations.

For pile foundations the following two failure criteria and corresponding limit states, are envisaged:

(1) the capacity of the most heavily loaded pile is exceeded, or

(2) the capacity of the entire pile system is exceeded after full redistribution of loads among the piles (i.e. fully utilized pile system)

Obviously, the criterion (2) leads to a higher load capacity. An important issue is how the redistribution of load carrying among piles take place, i.e. what the load-deformation characteristics are, and how large deformation would be involved in the redistribution of load carrying. Depending on the deformations, utilization of the systems capacity could have an effect on conductors which will interact with the jacket/piles in transmitting the loads to the seabed. Even if the load-carrying effect of the conductors is neglected, the deformation associated with the redistribution of loads will impose loads on the conductors that need to be considered in the design of conductors... Another matter is that the pile soil behaviour will have an effect on the forces in the (jacket) structure. Hence, it is important to consider the performance of the whole system in choosing limit states for the piles.

There exist several methods for determining e.g. the axial capacity of a pile. All of the methods calculate the total capacity as [31]:

$$Q_u = Q_{tip} + Q_{skin} \tag{6a}$$

where

$$Q_{tip} = q_{tip} A_{tip} \tag{6b}$$

$$Q_{skin} = \sum_1^n \tau_{skin,i} A_{skin,i} \tag{6c}$$

where Q_u is the ultimate pile capacity, Q_{tip} the tip resistance, Q_{skin} the skin friction resistance, q_{tip} the unit tip resistance, A_{tip} the gross pile tip area (calculated as $0.25 \cdot \pi \cdot D^2$ where D is the pile outer diameter), $\tau_{skin,i}$ is the unit skin friction resistance on pile segment i, $A_{skin,i}$ is the area of pile segment i, and n is the number of pile segments. For sand the material parameters are cone resistance, relative density, effective vertical stress and effective interface friction angle. For clay the material parameters are undrained shear strength, plasticity index, overconsolidation ratio, sensitivity, effective vertical stress and effective interface friction angle. However, the different methods use different material parameters. In addition, other factors such as the following need to be considered:

- Effect of combined axial, lateral and moment loading on the pile
- Effects of cyclic loading during a severe storm (see e.g. Figure 8)
- Effect of time after pile installation (ageing)
- Effects of erosion/gapping at the top of the pile

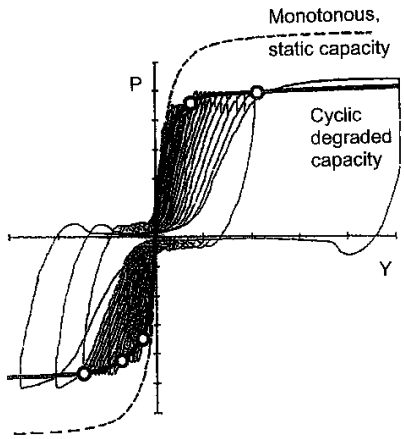
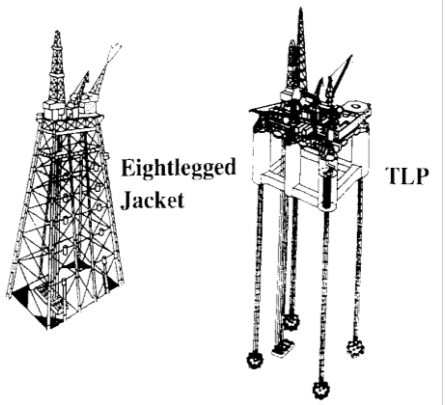
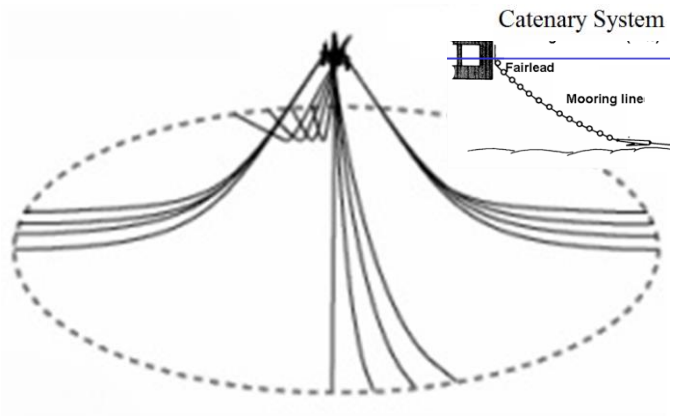


Figure 8. Cyclic degradation of soil.

The failure modes (Limit States) associated with gravity foundations include soil failure under sliding, and overturning (rotation).



a) Jacket b) Tether mooring system



c) Catenary chain system

Figure 9. Facilities amendable to systems analysis.

4.2.4 *Structural System Analysis*

Offshore structural systems can be categorized

- steel structures consisting of relatively slender unstiffened members and pile foundation
- structures (in steel or concrete) consisting of large diameter members (stiffened steel panels or shells, concrete plates or shells) – subjected to global and local pressure loads (typically floating steel structures; fixed or floating concrete structures)
- Monocoque structures, like ships, made of stiffened steel panels

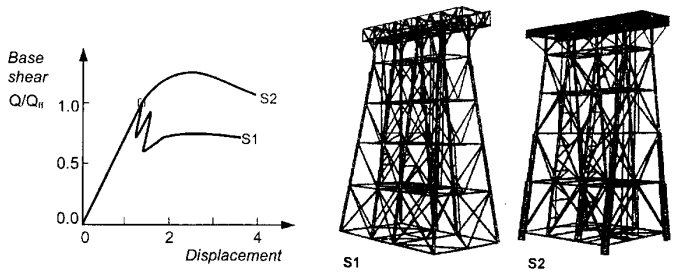
The limit states for systems in general need to be specified through

- a global instability limit state (as indicated in Chapter 2, or
- a set of sequences of component overload or fatigue failures by using a numerical method due to the complexity of the behaviour up to ultimate failure of a system.

Systems analysis is particularly of interest for systems subjected to accidental damage.

The systems in Figure 9 that consist of slender members, are amendable to nonlinear system modelling by a beam-column model of members or a tensile component, including the piles, and a shell or simplified model of the tubular joints, recognizing the possible fracture by using plastic strain or cross-section deformation criteria. See also comments in Section 4.2.2. In this way a true ultimate strength may be determined. The basic limit state for this structure is the ultimate global load the structure can carry, as shown e.g. in Figure7a. For a redundant structure the system strength depends upon the component characteristic (Figure 7a), as well as the system composition of components. Figure 7b shows typical behaviour of jackets under broadside static loading. The statically determinate K braced jacket fails in a "brittle manner" while the failure mode of the X braced structure is more "ductile", as well as yield a reserve strength beyond first member failure.

However, in this connection the presence of cracks and generation of cracks through large elastoplastic deformations should be accounted for. In addition redistribution of loads carried by different members might imply large displacements and hence impose "deformation" loads on risers and conductors. This issue will be particularly important in situations where the large deformation is concentrated to a small area, such as the upper storey without diagonal braces subjected to wave-in-deck loads. Unless it is demonstrated that the risers and conductors tolerate the imposed loads, a displacement limit for the global system needs to be defined and adhered to. Alternatively, the acceptability of the further consequences should be evaluated (For instance, a blowout risk may be controlled by DHSV, etc.). The question for the jacket S2 in Figure 10 is whether deformation of 2.5 m can have any adverse consequences.



a) Load-displacement Q_{ff}, is the base shear at first member failure b) Layout of jackets S1 and S2.

Figure 10. Global behaviour of jacket structures subject to broadside loading [32]

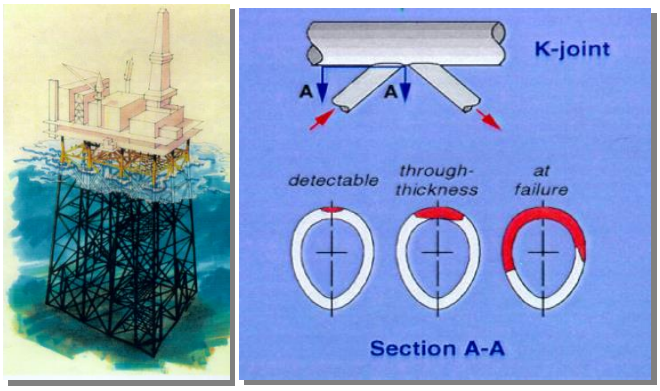
While the ultimate capacity of monocoque structures can be determined by FE shell elements, at least if the part that is undergoing failure can be limited to a few holds, .it is less feasible to do a nonlinear FE analysis of a frame structure consisting of large diameter members made up of stiffened steel panels or concrete shells. However, if such an analysis is conducted, fracture criteria both relating to loss of strength as well as flooding of buoyant members, should be considered. However, the special case of a floater (drilling platform) consisting of large diameter members made of stiffened panels and slender braces may be feasible for nonlinear analysis if it can be justified that the large diameter members would not fail and, hence, is modelled as elastic components and the nonlinear material and geometry analysis can be

focused on the braces and possibly the girder/truss in the deck structure.

4.3 Fatigue strength

Fatigue is a very local phenomenon, influenced by local geometry and weld defects induced by the fabrication process. In welded metal structures cracks normally start from fabrication weld defects with a depth of say 0.1mm. Cracks are driven by cyclic, stresses. Given an adequate inspection method and time, cracks can be detected and repaired. The mean detectable crack depth in Non-Destructive Examination is in the range 1-2 mm; while for Close visual inspection the mean detectable crack depth is 10-20 mm. This means that the weld quality cannot be completely ensured by inspection at the fabrication stage. Moreover, if cracks are not detected, they may reach a critical size and lead to fracture. Usually we differentiate between initiation, stable growth and unstable growth (fracture) of cracks, see [33, 34].

Cracks propagate in different ways depending upon the geometry. The jacket platform and ship represent two extreme cases. In a framework structure like a jacket the crack growth occurs in discrete joints and cause member failure. Subsequent failure may be ultimate or fatigue failure. In a monocoque ship structure a single crack can cause global failure by fracture. The definition of fatigue failure criterion implied in SN curves is visible crack; through thickness crack; or member failure. For plated joints with relevant plate thickness is applied while the width is small. In this case the difference between fatigue lives according to the various definitions is not large. However, for steel plated joints in a monocoque ship structure, the joint is surrounded by much material and the difference between fatigue lives according to the various definitions is large. Tubular joints are tested as model scale tubular joints and the crack growth differ significantly from that of plated joints (Figure 11). This implies also significant differences between the lives defined according to different criteria.



a) Jacket b) Crack propagation in tubular K joint

Figure 11. Crack behaviour in a jacket tubular K-joint.

High cycle fatigue criteria in design codes are typically expressed by SN curves; that is constant amplitude loading. Final fracture depends on the crack size and the load level. When the load is random, failure could occur at different crack sizes depending on the actual load level. In principle there is therefore an interrelation between fatigue and ultimate limit states. This is particularly an issue for tubular joints. For practical purposes it is therefore convenient to define both fatigue failure (under constant amplitude loading) and ultimate strength of the relevant components to limit this interaction and hence for separate FLS and ULS design checks.

Low cycle fatigue occurs in connection with severe cyclic loading causing repeated plastic deformations. This limit state has been of concern in connection with the low cycle variation of cargo loading on merchant ships ([33]) and ULS check of jacket structures under wave loads on a damaged structure to complement a nonlinear pushover analysis – in which the plastic capacity is utilized. The limit state for low cycle fatigue assessment is formulated as a numerical procedure with cyclic load history and modelling features of structure in [35].

As an alternative to empirical SN data, a fracture mechanics may be applied to formulate the fatigue limit state. Usually we differentiate between initiation, stable growth and unstable growth (fracture) of cracks. Fracture mechanics provide models for determining the two last stages by Paris Erddogan and CTOD/FAD methods, respectively [33] while the crack initiation time needs to be based on empirical data. While the fracture mechanics approach provides a tool for describing the development of cracks more in detail than the SN approach, it also requires more data, such as e.g. the initiation of crack size which are uncertain. However such data are implicit in SN data. Therefore fracture mechanics models should be calibrated against SN-data (e.g. [36]).

4.4 Effect of deterioration on ultimate failure

As indicated above fabrication defects might propagate in structures and lead to cracks that will reduce the ultimate strength. So will possible corrosion do. Similarly, cyclic loading on soil will lead to change of stiffness and strength and hence affect the ultimate strength. This degradation phenomena need to be accounted for by the definition of relevant limit states.

5 ROBUSTNESS AND LIMIT STATES FOR PROGRESSIVE FAILURE

5.1 General

Robustness or damage (fault) tolerance are desirable features of structures as spelled out in motherhood codes for structures, like [1-3, 37]. However, the formulations in these standards to a large extent reflect civil engineering practice. See also [38]. Even if the notion of robustness is not explicitly used in connection with buoyancy and (rigid body) stability of floating structures, it is also a most desirable feature in this connection [21].

For instance [3, 37, 39] describe robustness as “the ability of a structure to withstand events like fire, explosions, impact or the consequences of human error without being damaged to an extent disproportionate to the original cause”, thereby linking it explicitly to the concept of disproportionate collapse while recognising that total collapse is an acceptable outcome from a gross hazard. However, this issue needs to be resolved

by introducing more specific requirements relating to the probabilities of the scenarios in question. Sometimes, structural robustness is used to describe that the structure is insensitivity to local failure, in which modest damage (whether due to accidental or malicious action) causes only a similarly modest change in the structural behaviour; i.e. without referring to the disproportionate consequences. It is noted that the [40] states that accidents, misuse or sabotage are normally unforeseeable events and does not provide specifications on such actions nor design approaches. However, in a commentary to the ASCE standard it is recommended to limit the effects of local collapse – in a similar manner as in the other standards mentioned above.

The current version of [2] refers to a simplified version of the definition mentioned above, namely "the ability of a structure to withstand accidental and abnormal events without being damaged to an extent disproportionate to the original cause",

The definition of robustness implies or is synonymous with damage (or, generally fault) tolerance; i.e. that the damage should not escalate or progress into more serious consequences.

Moreover, damage tolerance is a crucial property for deteriorating structures to ensure the necessary safety when relying upon monitoring or inspection to detect cracks or corrosion damage, etc., and initiate the necessary remedial actions in case damage is detected. The cited definitions of robustness do not reflect this feature.

A Design, Inspection and Monitoring, Maintenance and Repair (DIMMR) approach is a fail-safety approach as opposed to safe-life design approach which aims at a low failure probability in the service life by design (and fabrication quality). The latter approach is typically used for systems that are difficult to inspect/repair of imply severe failure consequences. But the effect of IMMR depends upon the quality of inspection as well as the time required for repair. Hence, an inspection and repair measure can contribute to the safety only when there is a certain structural damage tolerance. This implies that there is an interrelation between design criteria (fatigue life, damage tolerance) and the inspection and repair criteria, see e.g. the comparison of the API and NORSOK fatigue design criteria for jackets with reliability based criteria in [6]. While the initial IMMR plan is made at the design stage, it is updated depending upon findings during inspections. DIMMR approaches were initially developed in the aerospace engineering (e.g. [41-43]), and nuclear engineering, but is now common practice in offshore engineering. However, the importance depends on environmental conditions, etc.

The focus on damage tolerance has especially been related to accidental actions – i.e. fires, explosions, vehicle impact for civil engineering structures. For offshore structures the considerations need to be extended to involve: ship collisions, dropped objects, abnormal ballast conditions and flooding due to structural damage or faulty pipes/valves, pumps, etc., or ballast operations. Moreover, it is necessary to include damage due to human errors in manufacturing or operation. Typical examples would be damage in terms of failure of mooring lines, failure of a slender brace in semisubmersible drilling rig (due to ship impacts, fatigue failure, e.g. [5]).

To conclude the discussion about robustness and damage tolerance it is necessary to define the concept to cover

- marine systems where relevant instability/capsizing and structural failure modes are considered
- damages caused by other phenomena than accidental loads (caused by operational errors); namely abnormal strength due to fabrication errors, etc Such damage needs to be assessed based on experiences. A typical example is the failure of one (or more) mooring line(s) or thrusters in DP systems.
- deterioration phenomena in view of fail-safe design approaches.

On this basis the following definition is suggested:
"the ability of a structure to limit the escalation of accident scenarios (floating positions; or structural damages - caused by accidental actions and abnormal strength due to fabrication or deteriorating phenomena - into accidental conditions with a magnitude disproportionate to the original cause"

Some efforts have been made to establish measures of robustness, e. g by [44, 45]. These kinds of measures are interesting but in practice a more explicit limit state – ALS or PLS - for verifying the damage tolerance is found to be more useful. Before presenting this approach, it is useful to place robustness issues in a perspective, by considering two accidents. Figure 12 shows the Ranger I jack-up in the Gulf of Mexico. One leg failed due to fatigue and the deck heeled. For such a system the overall reliability has to be ensured by adequate reliability of each leg. A second example is shown in Figure 12b. This is the Alexander Kielland platform. The brace D-6 failed due to fatigue and the other 5 braces connecting the column D to the platform failed in .condition with 6 – 10 m high waves. In the two mentioned accidents a failure of a member leads to catastrophic events. But, robustness is not necessarily synonymous with "redundancy". Consider for instance the single column platform shown in Figure 12c. Clearly this structure does not score highly in terms of redundancy. However, for relevant damage scenarios, the reserve capacity of a thick-walled large diameter concrete column, after relevant damage scenarios, is .significant.

The ALS check is directed towards avoiding global collapse, but also failure of safety systems like evacuation/ escapeways which are crucial for limiting failure consequences in terms of fatalities.

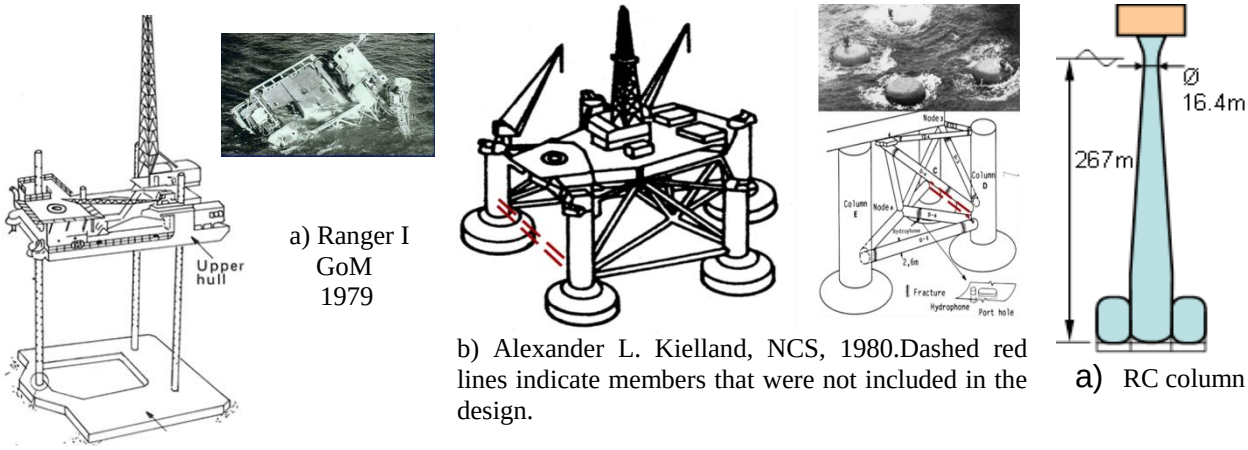


Figure 12. A perspective on robustness of offshore systems.

5.2 Design to prevent progressive collapse

5.2.1 *General*

While robustness or damage tolerance have been specified as desirable properties of structures or engineered systems in general for decades, there have been limited explicit criteria on how to implement such criteria in practice.

Accidental Collapse Limit State (ALS) criteria are introduced to limit the corresponding residual risk i.e. to prevent progressive failure. The basic principle relates to the fact that accidents develop in a fault sequence of events and it becomes important to establish a barrier to stop the escalation of the accident. This goal could be achieved by (e.g. [1]) by either of the following approaches

- designing the structure locally to sustain accidental actions and other relevant simultaneously occurring actions. (key element design); i.e. a quantitative "ULS" approach for designing elements, the removal of which would lead to a collapse defined as disproportionate, for an accidental load case. This is a component design check. This method will normally imply higher structural costs than the systems approach outlined below since all parts of the structure that can be subjected to accidental actions, need to be designed for such actions.
- designing the structure by accepting local damage but require the damaged structure to survive relevant actions (alternate path design). The relevant damage may be obtained as the effect of accidental actions. Systematic experiences from such analyses may serve as basis for specified damage conditions, representative for a certain industrial environment. In addition, "damages" implied by fabrication errors need to be considered. Such damages normally need to be specified by judgement. This will be a system design check.
- Designing the structure to meet robustness requirements through (prescribed) minimum levels of ductility, continuity and tying (Tie-force based design methods):

Clearly, all these approaches are applicable to structural integrity. In practice the first two methods are implemented. The third method, however, relating to ductility and continuity is also crucial in making the second method work.

The first method is, by its character, only applicable for structural strength. It is noted that the Eurocode refers to this method as a ULS check in a similar manner as for other loads; i.e. with a set of load combination scenarios involving accidental events.

The current version of ISO 19900 does not clearly present the second method either. This fact implies in a way that the first method which is close to the conventional ULS method, should be applied.

This could work well for e.g. explosion actions and dropped objects. It is more difficult to see how it can be applied in case of fire and ship impact scenarios. The combination of an (accidental) fire action – primarily implying a strength reduction, and other actions, with appropriate action and resistance factors, could be used. In connection with the limit state relating to ship impact, both strength and puncturing of components which is important for maintaining buoyancy, should be considered. One (conservative) approach could be to use a strength design philosophy for ship impacts (i.e. with energy absorption in the ship), see Figure 13, respectively.

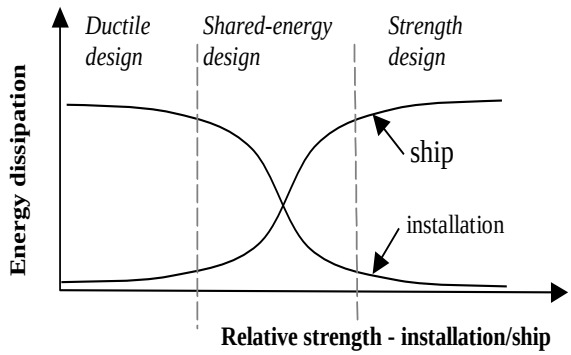


Figure 13. Ship impact design principle based on relative energy sharing between ship and installation [46].

However, the first method does not seem to allow for account of damage tolerance for situations where the damage cannot be established based on accidental actions but is specified based on experience, such as for station-keeping systems. Moreover, it is not applicable for damage tolerance checks relating to (rigid body) instability.

The second method was initially made a regulatory requirement in [14] and is currently specified in NORSOK N-001 [47]. It is applied to ensure damage tolerance in view of global structural, mooring a foundation failure as well as instability see Figure 14. The damage is to be determined by a risk assessment.

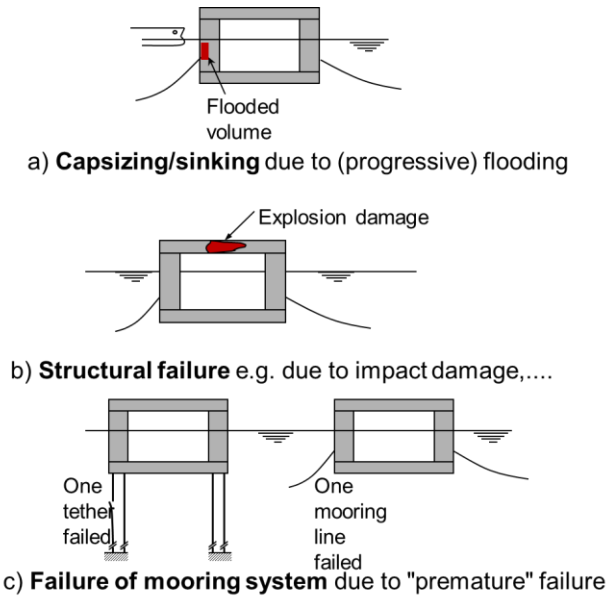


Figure 14. ALS design check for different failure modes.

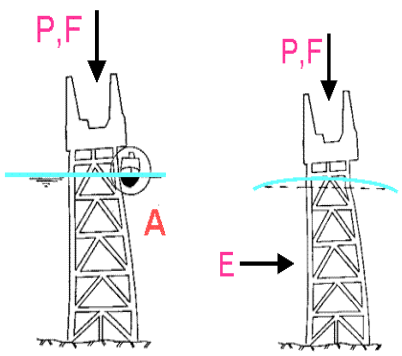


Figure 15. Accidental Collapse Limit States for different global failure modes, considering accidental (A), Environmental (E),.functional (F) permanent (P) actions.

The structural integrity criterion in NORSOK N-001 is expressed by a two-step procedure as illustrated in Figure 15, based on characteristic actions and resistances. The first step is to estimate the initial damage due to accidental actions or damage conditions (caused by human error) with an annual exceedance probability of 10^{-4}. This exceedence probability refers to accidental events on the whole platform and needs interpretation, as discussed by [15]. The second step is to demonstrate that the damaged structure resists relevant functional and environmental actions with a characteristic value depending on their correlation with the event initiating damage – without global failure. The characteristic resistance value used for steel is defined as a value exceeded with a probability of 95%. Load and resistance factors for steel structures are taken to be 1.0 in these design checks.

The NPD/NORSOK approach is applicable to the other failure modes, like rigid body instability, station-keeping system failure.

Moreover, the potential influence of human errors on fatigue failure needs to be accounted for partly by ALS design check but also in the IMMR. Moreover, a large fatigue design factor (FDF) will be an efficient risk control measure since it will reduce the stress level and hence crack growth rate and give more time for detecting and repairing cracks.

To estimate damage, i.e. permanent deformation, rupture, etc., of parts of the structure, nonlinear material and geometrical structural behaviour need to be accounted for. While in general nonlinear finite element methods need to be applied, simplified methods, e.g. based on plastic mechanisms, are developed and calibrated using more refined methods, to limit the computational effort required.

A particular issue is how the accidental actions or damage is determined; e.g. prescribed or based on risk analysis. Risk-analysis for offshore facilities is outlined e.g. by [48]. However, it should be noted that extensive experiences with accidental actions for typical platforms have lead to specific actions [15].

Often it is suggested to ensure robustness by redundancy considerations; e.g. require survival after removal of individual components. However, the component can mean a 15 m diameter concrete cylinder with a wall thickness exceeding 0.5 m, a stiffened steel cylinder with plate thickness in the range 20-40 mm., a an unstiffened steel cylinder with diameter and plate thickness of 2 m and 30 mm, respectively and a chain or steel wire with diameter 100 mm. Clearly, the vulnerability of such components vary. Moreover, it varies with the location of the component relative to the spatial variation of the hazards. The damage in many cases will not be a failure but partial failure of a component. These facts suggest use of a rational approach based on risk assessment. But also in this connection experiences should be used to specify damage conditions. The damage for floating structures would have to refer to its effect on structural integrity as well as floating ability/stability. In connection with stability a risk assessment approach will be an extension of the conventional method based on prescribed flooding of 1 or 2 compartments since the damage will depend on the material and layout of the structure and also account of all hazards causing flooding, including error in ballast operations. Moreover, it is noted that damage to the “submerged” parts of a floating structure, leads to a change of the floating position which hence will influence the wave and current actions on the structure.

Another issue is how normal uncertainties are dealt with. More sophisticated probability-based approaches in which some sort of quantitative or semi-quantitative model is constructed are now being developed. These typically seek to establish a given level of reliability in the structure, i.e., to demonstrate that the probability of failure is less than some defined threshold. Such methods are not currently implemented in codes and standards, although [39] does contain an annex which sets out a probability-based framework which may be used if required.

Clearly, catastrophic accidents might be caused by extreme environmental and any type of actions. The different criteria need to be balanced with respect to a defined acceptable risk level. For instance, the initial ALS criteria, based on accidental actions with an annual exceedance probability of 10^{-4} were said to imply a probability of total loss of 10^{-5}. The question is also what kind of consequences an action with probability 10^{-5} will cause.

Moreover, a large FDF will be an efficient risk control measure since it will reduce the stress level and hence crack growth rate and give more time for detecting and repairing cracks.

Finally, it is noted that ISO 19900 refers to Accidental Collapse Limit State (ALS) for the limit state in question. When it first was introduced (by NPD) it was called Progressive collapse Limit State (PLS). The name should reflect the content. Currently, the intention of ALS is to

- prevent escalation of an accident (damage) essentially caused by human errors resulting in accidental actions or abnormal resistance or deterioration of resistance
- ensure an acceptable risk level relating to functional and environmental actions

The main issue here is to achieve an acceptable risk level. In the following I would refer to probability values used by NPD (now NORSOK N-001 [47]), but other values might be used in other regulatory regimes. When the formulation was introduced in NPD [14] with accidental actions referring to an annual exceedance probability of 10^{-4}, an implied annual probability of total loss of 10^{-5} associated with each hazard, was intended [5]. This is the basis for the first part of the first intention – relating to accidental actions and damages. The second part of the first intension materialized by relating the fatigue design criteria to the consequence of fatigue failure; i.e. making the FDF dependent on whether the structure satisfied ALS criteria or not after fatigue failure of relevant "member" (in this connection it should be noted that there could be a difference in fatigue failure (through thickness crack) versus member failure. (It is noted that the consideration for monocoque structures - like ships - would be different since a crack might be propagating until final hull girder rupture).

The second intention appeared partly because it could be considered a special case of the two step ALS approach where the environmental actions with a probability 10^{-4} is used in the first step and omitting the second step because it refers to the same environmental loads but with a lower magnitude. Another important issue is that a consistent way of determining wave actions should be based on the relevant wet structure. Then there will be a significant difference between, say, the 10^{-2} and the 10^{-4} wave action – especially in view of wave-in-deck, depending on air-gap requirements applied. However, action factors and analysis methods to demonstrate compliance are also different.

It is clear that PLS would be a better term for the limit state with the first intention mentioned above criteria. For the second intention PLS could probably be justified by the fact that the documentation of "survival" in the second step might involve a development of global failure by gradual "component failures".

5.2.2 *Analysis methods for determining the accidental damage and resistance for the damaged system*

To demonstrate compliance with ALS requirements calculation of the damage due to accidental actions as well as the ultimate capacity of a damaged structural system is needed. To estimate damage, i.e. permanent deformation, rupture etc. of parts of the structure, nonlinear material and geometrical structural behaviour need to be accounted for. Dynamic effects may be of importance for explosions and ship impacts. Recent advances in computer hardware and software have made nonlinear finite element analysis (NLFEM) a viable tool for assessing damage and system resistance for steel structures. Examples of general purpose computer codes, which have been used widely are ABAQUS, ANSYS and LS_DYNA. Specialised software is available for particular tasks.

Simplified methods based on plastic analysis often provide fast and amazingly accurate estimates of the damages caused by accidental actions on steel structures (e.g. NORSOK N-004 [46]) and are especially useful in early design for screening purposes. In particular cases where simplified methods have not been calibrated, nonlinear time domain analyses based on numerical methods like the finite element method should be applied.

In finite element analyses of collisions a careful choice of mesh is required in order to obtain accurate results, especially for components deforming by axial crushing. A major challenge in NLFEM analysis is prediction of ductile crack initiation and propagation. This problem is not yet solved. Crack initiation and propagation should be based on fracture mechanics analysis, using the J-integral or Crack Tip Opening Displacement method rather than simple strain considerations. The simplest approach to the problem is to remove elements once the critical strain is attained. This is fairly easily done in an explicit computer codes to treat the transient dynamic problem, because there is no need to assemble and invert the effective system stiffness matrix. However, deleting elements disregards the fact the large stresses can be maintained parallel to the cracks. An improved modelling is to introduce a double set of nodes such that the elements are allowed to separate once the critical stress is attained. A drawback with a double set of nodes is that the potential location of cracks needs to be defined prior to analysis.

The assessment of the true ultimate strength of the global structure requires nonlinear system analysis. Such an approach has been developed and validated for framed structures consisting of slender members. However, a particular challenge is the nonlinear behaviour of the tubular joints. Hence it is also important not only to check the capacity in a pushover mode but also carry out analysis with a representative cyclic storm loading [49]. Even if general purpose nonlinear FE methods might be used to deal with the ultimate structural behaviour of platforms with large diameter columns and pontoons and ships, limited efforts have been devoted to such analyses, partly because global strength is not as critical as local damage causing flooding and instability of such structures.

Compliance with the global strength requirement of the damaged structure can in some cases be demonstrated by

removing the damaged parts, and then accomplishing a conventional ULS design check, based on a global linear structural analysis and ultimate strength checks of components. Such methods may be very conservative, especially for damaged structures. Fixed platform analyses are carried out by modelling the pile-soil behaviour by equivalent linear or nonlinear concentrated springs or, distributed springs along the piles, or continuum (finite element) model that represent stiffness and foundation, capacity, appropriately using the material properties in the different soil layers. Soils exhibit nonlinear behaviour, even at low load levels, which needs to be accounted for. Software dedicated for progressive collapse analysis of frame offshore structures have also been developed, e.g. USFOS and SACS [50].

CONCLUSIONS

General

As an introduction the framework for safety management of offshore structures is outlined in terms of a life cycle approach involving design (including planning of fabrication and operation), and QA/QC in all phases and especially inspection and monitoring during fabrication and operation. The focus herein is on formulation of limit states for all relevant failure modes that facilitate a basis for serviceability and safety criteria for design and follow-up during operation. Limit states are used together with characteristic values or probabilistic models of actions and resistance variables to obtain a measure of safety in the design. The definitions of characteristic values or probabilistic models can vary between jurisdictions and are not pursued in this context.

The following types of limit states are currently used:

- ultimate and progressive development instability of rigid bodies (ULS, ALS1); as well as
- ultimate, fatigue and progressive failure limit states (ULS, FLS, PLS –ALS for structures and station-keeping systems is summarized. In particular the background for robustness or damage tolerance requirements is highlighted and how limit states can be formulated in terms of ALS criteria.

Efforts to establish measures of robustness are briefly commented upon. Such measures have been established for jacket structures e.g. in terms of a deterministic measure based on the ratio of the global strength with and without damage. For a given structure it will then be a conditional measure depending on the magnitude and location of damage as well as the spatial action variation. This kind of measures are interesting, but in practice a more explicit limit state for verifying the damage tolerance is discussed herein.

Considerations for ISO 19900

The following aspects are important to consider in the revision of ISO 19900:

Limit states should preferably

- be based on structural mechanics theoretical formulations (not pure regression to data)
- explicitly contain the parameters of influence
- have as a small (random) model uncertainty as possible
- be as simple as possible
- be feasible to analyse.

Current safety limit states include ULS, FLS, and ALS. It is suggested to keep these limit states; i.e. not only refer to ULS only for safety. As outlined below the limit states can be made more precise. .Moreover, it is suggested to reconsider the term accidental collapse limit state in view of the alternative: progressive failure limit state (PLS) (as it was initially called when introduced by NPD). Moreover it is important to harmonize the criteria for instability of floating structures with the structural engineering approach; and especially the implied risk level – which is beyond the scope of the present study. In this connection it needs to be recognized that both ISO and IMO have interest in establishing design codes for floating structures.

Stability limit states are formulated in terms of forces or actions only and involve

- Overturning of a rigid body resting freely on the seabed or the structure
- Instability (capsizing) of floating structure (with catenary mooring or Dynamic Positioning)
- Stability of tension-leg platform (slack or failure of tethers)

The fact that stability criteria are expressed by actions implies that partial (action) factors need to be larger and smaller than 1.0 if they are unfavourable and favourable, respectively.

It is noted that the framework for floating structures differ from structural strength. However intact and damage stability requirements correspond to ultimate and accidental collapse limit states, respectively.

It is noted that there is a connection between deterioration phenomena and ultimate failure – for structures and soil. This fact needs to be observed when defining the respective limit states for ULS and FLS for structures and ULS for soils.

Ultimate strength formulations for structural components are determined by experiments and numerical analysis. The limit state for strength is based on strength, but plastic strain or equivalent cross-section deformation criteria should be accounted for since these conditions might imply cracks that can propagate due to fatigue. This issue is particularly important for tubular joints.

Similarly, fatigue limit states (based on SN data obtained by constant amplitude testing) should be based on a conservative failure criterion – due to the risk of premature fracture due to overload in a real random loading condition. Obviously, the considerations of ULS and FLS limit states need to be balanced.

As an alternative to the SN-method crack development might be modelled by fracture mechanics, in which the initial crack size is based on empirical data while the stable growth and final rupture is modelled by fracture mechanics. Due to the uncertainty of the data, especially the initial crack size, the fracture mechanics approach needs to be calibrated against the SN data for fatigue failure, which are obtained in laboratory tests of specimens with full scale thickness and somehow represent the true physical behaviour up to fatigue failure.

The common definition of fatigue failure refers to a visible or through the thickness crack and that there is a reserve crack growth between this limit state and a state corresponding to member failure, and especially ship hull girder failure. On the other hand, in large monocoque structures there will be multiple crack sites that can lead to coalescing cracks.

The limit state for individual piles refer to its ultimate capacity based on skin and tip resistance, which depends on the effect of cyclic actions during a severe storm, time since pile installation and erosion/gapping at the top of the pile. Alternatively the limit state might be based on the capacity of the entire pile group – assuming sufficient "ductility"/compliance that a fully utilized capacity can be developed. Moreover, it is then important to ensure that the redistribution of loads between piles can occur with limited deformations, which could impose actions on e.g. the jacket or conductors. Anyway, a global model including the structure and pile foundations needs to be used in this analysis.

The ultimate capacity of the supporting structure is of major importance. To estimate the ultimate capacity a nonlinear material and geometrical analysis is desirable. Such methods are already in use for framework platforms consisting of slender members (jackets, jack-ups) and pile or bucket foundations. In order to represent the global ultimate limit state it is important that this analysis is based on a consistent nonlinear model for the structure and the pile-soil foundation as well as reflects the following features that will affect what should be considered as the ultimate capacity:

- local buckling of members, fracture in tubular joints
- global deformations that can impose deformation actions on e.g. risers, conductors

When the ultimate capacity is utilized by accounting for elasto-plastic effects to accommodate redistribution of load sharing between components, it is also important to validate the global limit state by a "low cycle"/shakedown analysis for a storm condition.

Regarding limit states for individual mooring lines ultimate and fatigue limits refer to "member" failure expressed by the line tension. Experiences show that local effects at fairlead, anchor, etc., need to be more explicitly accounted for.

In connection with the global or system limit state, robustness and damage tolerance are important features. Emphasis is placed on establishing a general procedure applicable for the hull and mooring system, considering failure modes involving structural strength and rigid body stability. Damage stability requirements were introduced for ships in the SOLAS Convention in 1948 after a lengthy process catalysed by the Titanic accident in 1914, and appeared in the first codes for floating platforms around 1970, explicit damage tolerance criteria for buildings were introduced after the Ronan Point accident in 1968. Damage tolerance criteria for the strength of hull structures and mooring lines were introduced after the Alexander Kielland accident [16] in 1984.

The following three approaches are commonly proposed to provide robustness in terms of structural strength:

- Designing the structure locally to sustain accidental actions and other relevant simultaneously occurring actions. (key element design); i.e. a quantitative "ULS" approach for designing elements, the removal of which would lead to a collapse defined as disproportionate, for an accidental load case. This is a component design check.
- Designing the structure by accepting local damage but require the damaged structure to survive relevant actions (alternate path design). The relevant damage may be obtained as the effect of accidental actions or prescribed on the basis of experiences. The Accidental Collapse Limit State (ALS) is intended to represent this approach.
- Designing the structure to meet robustness requirements through (prescribed) minimum levels of ductility, continuity and tying (Tie-force based design methods):

The third method is in a way implicit the first two methods. The second method is the most general and is actually also used for damage stability check of floating platforms and station-keeping systems. The first method can be applied for explosion actions but is more difficult to implement for e.g. fire and ship impact scenarios, but is normally expected to be conservative.

An important issue in connection with this ALS method is how the damages – with respect to strength and leak into buoyant components - are determined, especially to which extent risk assessment methods are applied. Sometimes, failure (removal) of a component (member) is used as damage. Obviously this is a simplistic approach not generally applicable for components that vary from thick.-walled, large diameter RC cylinders to steel wire for mooring lines. Often the damage is partial.

The second method is not explicitly formulated in the current ISO 19900 and there is limited reference to rigid body stability.

Regarding the mooring or DP system both strength and drift-off/motion limits are relevant.

Finally, it is recommended to reconsider the definition of the following terms: Accidental vs. progressive collapse Limit State; Action classification (depending on source, probability or occurrence, etc.), Damage tolerance; Robustness; Vulnerability.

ACKNOWLEDGMENTS

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17.2 Presentation by Philip Smedley

NTNU Norwegian University of Science and Technology

International Association of Oil & Gas Producers

Special Session:
Development of the Third Edition of ISO19900

SC7/WG1/TP2 Risk
(Risk, Consequence, and Reliability Classes/Targets)

Philip Smedley, BP

The 3rd Offshore Structural Reliability Conference
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TP2 Risk - Scope

- Risk characterization and life safety issues
  - Modify current L1, L2, L3 (exposure)?
  - Risk classes affecting Quality Control
  - Alignment with API and other standards?
  - More material from 19902 and 06 into 19900?
- Reliability "targets"
- Targets for service life extension

TP2 Risk - Members

- Philip Smedley, BP (lead),
- Moises Abraham, Chevron,
- Michel Birades, TOTAL,
- Mike Hoyle, DNV GL ND,
- John Waegter, Ramboll,
- Dave Wisch, Chevron and API 2GEN link,
- Andrea Mangiavacchi, Experia Consultant
- Graham Thomas, Consultant WG8 Arctic link (since July)
- Richard Snell, Ret'd BP and former SC7 Chair (since July)

TP2 Risk - Activities

1. Seek industry feedback on ISO 19900 from TP members, their Companies and Country members, & other ad-hoc Contacts : especially on Exposure Levels
2. Review feedback, ISO 19900 Cl 6, ISO 19901-9 SIM draft, and agree way ahead for new Cl 6 Exposure Levels (if still desired)
3. Draft new Cl 6 text and share with
   1. TP2 members, WG1 Convenor and other TP leaders
   2. WG Convenors & Project (Standard) Leaders
   3. Wider industry at OSRC#3 in Stavanger
4. Finalise Cl 6 text – open for comment during DIS review
5. Plan for updates to text around reliability, QC, etc.

TP2 Risk – Exposure Levels (Cl 6)

1. Current 19900: 6 pages
   - 3 Life-safety categories S1 – S3
   - 3 Consequence categories C1 – C3
   - 3x3 matrix giving 3 outcomes Exposure Levels L1 – L3

Table 1 — Determination of exposure level

Life-safety Category	Consequence category		
	C1: High consequence	C2: Medium conse-quence	C3: Low consequence
S1: Manned non-evacuated	L1	L1	L1
S2: Manned evacuated	L1	L2	L2
S3: Unmanned	L1	L2	L3

2. Draft 19901-9
   Support text very similar to 19900

Table 1 - Minimum Fitness-for-service Performance Level

Life Safety			Environmental / Financial		
Consequence	RP [1]	Comments	Consequence	RP [1]	Comments
Manned	2 500 [2]	Performance level may be relaxed for manned-evacuated platforms	High	1 000	
Unmanned	N/A	No fatalities.	Low	100	Limited environmental consequences.

TP2 Risk – Exposure Levels (Cl 6)

High level opinions:

1. Simplify language throughout 19900 series, where practical
2. Strong preference to keep different requirements for different potential consequences
3. Three levels seems about right
4. While top level should capture any life safety risk, the term ‘manned’ may not be optimal.
5. If we are going to differentiate do so consistently - some Standards only cover L1, others cover all levels.

TP2 Risk – Exposure Levels (Cl 6)

Outcome:

1. Keep L1 – L3
   1. No 3x3 matrix but similar categorisation
   2. L1 Life safety, L2 Env. risk, L3 Business risk
   3. Biggest difference is “unmanned but high env consequence” moves from L1 to L2.
   4. No notable impact on other 19900-series standards.
2. Simpler text to explain categorisation (2.5 pages)
3. Some background information in Annex (1.5 pages)
4. What about API 2GEN DIS update?

TP2 Risk – Reliability/QC

Recommendations for ISO 19900:

1. Minimal changes in normative text.
2. Provide some more explanatory text in Annex A around 19900-series philosophy / calibration history
3. No target reliability values (Smedley keynote)
4. Point towards OSRC#1 IOGP report for guidance on min reliability levels for specific structure types.
5. Ted Fletcher of Woodside has some opinions for improving Cl 11 Quality Management
6. Challenge over philosophy around reliability of new build vs existing structures (Assessment) – TP4

17.3 Presentation & article by Paul Frieze, A. Morandi, R. Snell, T. Moan & M. Maes

NTNU Norwegian University of Science and Technology

International Association of Oil & Gas Producers

Special Session:
Development of the Third Edition of ISO 19900

SC 7/WG 1/TP 3 Uncertainty Assessment

Paul Frieze, PAFA Consulting Engineers

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TP 3 Scope & Membership

- Singapore January 2016 WG 1 project
- Technical Panel 3 tasks
  - New annex on uncertainty assessment
  - Impact on any normative clauses in the main Standard
- Members - appointed
  - Paul Frieze, PAFA Consulting Engineers (lead)
  - Sathish Balasubramanian, ExxonMobil
  - Torgeir Moan, NTNU
  - Marc Maes, Aleatec AS Inc.
- Members – co-opted/assigned
  - Alberto Morandi, American Global Maritime
  - Richard Snell, Oxford University

TP 3 Uncertainty Assessment - Activities

- Reviewed the requirements of ISO 2394:2015, *General principles on reliability for structures* with particular emphasis on application to the proposed ISO 19900 Annex on *Uncertainty Assessment*
- Proposed a Contents List for the Annex
- Sought additional members for the TP
  - ❖ Alberto Morandi
  - ❖ Richard Snell
- Participated in WG 1 telecons
- Prepared the OSRC paper

TP 3 Activity – Review of ISO 2394

- ISO 2394:2015, *General principles on reliability for structures*
  - ➢ Has 3 different definitions relating to *uncertainty*
  - ➢ Entire clause (6) devoted to *Uncertainty representation and modelling*
    - Clauses 6.1 *General* and 6.2 *Models for structural analysis* – comprehensive
    - Clauses 6.4 to 6.6 – relatively brief
- In contrast, ISO 19900 has no definition of *uncertainty* and the term only appears 12 times
- Many ISO 2394 provisions on *uncertainty* are directly relevant to ISO 19900 but, given the extensive use of SRA in the offshore industry, ISO 2394 seems to work at a simpler level than is usual in offshore applications

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TP 3 Activity – Review of ISO 2394

Highlights:

- Treats environmental and seismic actions as variable actions although an infrequent earthquake is an accident but not infrequent metocean conditions, i.e. there is no such thing as an abnormal action
- The *calibration* procedure for deriving partial factors does not align with that used in the API/Moses work involving existing exemplary structures
- It seems to believe that the representative value of steel yield strength is a characteristic value (5%) whereas it is always a nominal code value
- 6.4 *Model uncertainty*, at first reading, seems to be describing *modelling uncertainty parameter*, a statistical parameter used in SRA studies to define the accuracy of a codified strength equation (model). However, it actually appears to be suggesting that each basic variable in the strength equation has its own modelling uncertainty parameter which is quite different from the one normally used in SRA studies.

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TP 3 Activity – Review of ISO 2394

Highlights:

- Annex D (informative) is devoted to *Reliability of geotechnical structures*
- Provides guidance for the application of the normative requirements of ISO 2394 to geotechnical design using either Clause 8 *Reliability-based decision making* or Clause 9 *Semi-probabilistic method* which includes the *Partial factor method* (as adopted in the ISO 19900 Series of Standards)
- Good introduction to the full range of issues needed to be addressed when trying to quantify uncertainties in a both spatially and temporally varying medium
- When simplified a little, is directly amenable to application to steel and concrete structures and, to some extent, overcomes the above criticism concerning application of ISO 2394 to offshore structures
- Needs serious editing to define all the parameters used and to achieve consistency with the normative text

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TP 3 Activity – What is "uncertainty"

Highlights:

- ISO 2394 does not make clear what is meant by uncertainties - example

  9 *Semi-probabilistic method* 9.2 *Basic principles*

  Introduces resistance R using the following general model

  $R = b\ \theta R(X, a)$ (17)

  X are the material properties

  a are the geometric parameters

  θ is the model uncertainty related to the resistance model

  b is the bias in the resistance model.
- Partly clarified at D.4 *Statistical chacracterization of model factors*

  Model factor M defined as ratio of "*measured to the calculated capacity*"

  $M = Q_m/Q_p$ (D.1)

  Q_m is measured capacity

  Q_p is predicated capacity

  Active SRA users would immediately recognise this as "modelling uncertainty parameter" and know that it is defined in terms of a bias, a standard deviation or COV, and a statistical distribution. Reference to "*statistical assessment*" in the following paragraph would suggest this is the intended scope of the term "uncertainty"

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TP 3 Activity – What is "uncertainty"

Defining "uncertainty"

- The ISO 2394 definition of "uncertainty" is ambiguous.
- A layman's interpretation would probably be limited to "standard deviation" or "COV", i.e. no bias and certainly no associated distribution
- Personally I am not keen to redefine terms that have a relatively clear "lay" meaning with an ISO standard definition. A particularly good example is the term "risk" because without very strict editing of standards, the lay use appears not infrequently

Uncertainty is meant to encompass bias, standard deviation and probability distribution

- In the context of ISO 2394, "statistical modelling" seems a more appropriate phrase to describe the determination of probabilistic properties of bias (or mean), standard deviation (or COV) and distribution
- The statistical modelling is to covers basic variables as well as modelling uncertainty parameters (or modelling factor according to ISO 2394 Annex D) although in its general sense, variable could also include modelling uncertainty parameter
- Instead of ***Uncertainty modelling***, suggest ***Variable statistical modelling***

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TP 3 Contents List for Annex

1. Introduction

- Make maximum use of ISO 2394 but make clear where it does not apply because of preferred/established techniques used in offshore SRA studies

2. Uncertainty/Variable statistical data

- Database of uncertainty data - to be seen as a repository of (standardised) uncertainty data
- Ensure adequate QC procedures are in place and are applied to data before entry into database. Identify some appropriate procedures, e.g. database screening approaches used in creating ISO 19902 tubular member and tubular joint databases

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TP 3 Contents List for Annex

2. A. Material properties
   - i. Steel
     - a. Yield strength and tensile strength
     - b. Charpy impact tests
     - c. CTOD
     - d. CE and modified CE
     - e. SCFs
     - f. Fatigue tests and S-N curve, endurance limits
     - g. ...
   - ii. Concrete
   - iii. Soils
     - a. Cohesive - undrained shear strength
     - b. Cohesionless – skin friction factor, end bearing factor, limiting values
     - c. ….
   - iv. Synthetic fibre
   - v. …

   B. Component strengths
   - i. Tubular members and joints
   - ii. Other components
   - iii. Piles
   - iv. plates and stiffened panels
   - v. stiffened cylinders
   - vi. chains, tendons, cables, ropes, ...
   - vii. ...

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TP 3 Contents List for Annex

2. C. Component fatigue
   - i. Tubular joints
   - ii. Other components
   - iii. Ship, semi-submersible & spar details
   - iv. chains, tendons, cables, ropes
   - v. …

   D. Metocean
   - i. hindcast data – regionally-based - short-term and long-term wave height distributions and heights in an interval T based, e.g. on Tromans and Vanderschuren methodology (TVM)
   - ii. measured data
   - iii. ...

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TP 3 Contents List for Annex

3. Uncertainty calculation techniques & procedures

- Describe the techniques and procedures to be adopted when determining the properties, strengths, metocean parameters, etc. They need to be clear and unambiguous to minimise misinterpretation.

A. Material properties
- i. Steel
  - a. Yield strength and tensile strength – determined in accordance with standardised procedures. Record strain rate
  - b. Charpy impact tests - see ISO 19902 20.2.2.4
  - c. CTOD - See ISO 19902 Annex B
  - d. CE and modified CE – see ISO 19902 20.2.2.4.2
  - e. SCFs
  - f. Fatigue tests and S-N curve, endurance limits
  - g. ...
- ii. Concrete
- iii. Soils
  - a. Cohesive - undrained shear strength
  - b. Cohesionless – skin friction factor, end bearing factor, limiting values
  - c. ….
- iv. Synthetic fibre
- v. …

B. Component strength
- i. Screened databases – Experimental results
- ii. chains, tendons, cables, ropes, ..
- iii. …

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TP 3 Contents List for Annex

4. Example calculations

A. Component strengths

Describe the various sources of tubular member strength data and why particular test series were deleted from the database.

B. Maximum wave heights

Describe application of TVM to hindcast data for determination of maximum wave heights in an interval T.

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TP 3 Contents List for Annex

5. Gross errors, Human factors, Equipment performance

These topics represent those not covered by SRA or directly by structural design standards. However, as a source of hazards, they can be more important than structure especially for floating platforms.

ISO 2394 addresses some of this through its robustness requirements, based on risk assessment. It introduces a robustness index and other robustness measures. One of these, defined as *Residual Influence Factor* (wrongly attributed to ISO 19902) exploits RSR via

$RIF_i = RSR_{fail,i}/RSR_{intact}$

RSR_{intact} is *RSR* of intact structure

$RSR_{fail,i}$ is *RSR* with member *i* either failed or removed

At least one major oil company has had a process for many years to derive RIF_i for a fixed steel structure by automating USFOS to remove every member/joint in turn – an overnight exercise which provides the complete list next morning

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TP 3 Impact on Normative Requirements

- For ISO 19900, direct impact of *variable statistical modelling* is likely to be minimum other than to improve consistency of terminology – similarly for other ISO 19900 series standards
- However, given the need, particularly for floaters, to ensure robustness, implementing even a modified version of the ISO 2394 requirements is likely impact on ISO 19900 normative requirements
- Because *variable statistical modelling* data is mostly used in SRA studies, this will help generate more consistency when such studies are performed. However, without a documented approach for executing SRA, real consistency will be hard to achieve
- Generally, it is likely to lead to more consistent application of procedures such as TVM because these are fully not documented as yet

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Uncertainty Assessment in ISO 19900

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ABSTRACT: Uncertainty assessment is generally the responsibility of code writers as the challenge to quantify these can be complex. The policy of ISO/TC 67/SC 7 in preparing its suite of offshore structural standards was to minimise the extent to which users would need to conduct such assessments. At present, 'uncertainty' is not defined in ISO 19900, however, its main superior standards, ISO 2394, not only defines uncertainty but also devotes an entire clause to its representation and modelling. However, a critique of ISO 2394 reveals that it works at a more basic level than is commonly practiced on offshore structures. Some suggestions are made for the supplemental information and guidance needed in ISO 19900 in order that ISO 2394 can be exploited for offshore structure application. Sources of structural strength uncertainty are identified and some relevant values listed covering fixed and floating structures: they primarily address steel structures in non-arctic environments. The effects of gross errors are discussed and many arise from non-structural subsystems: some of these are identified. Robustness is seen as one way of dealing with gross errors and ISO 2394 seems to provide some useful measures of these. For the proposed ISO 19900 annex on Uncertainty Assessment, a possible list of contents is detailed.

KEY WORDS: Uncertainty; modelling uncertainty parameters: fixed steel structures; floating offshore structures; gross errors; robustness.

1 INTRODUCTION

Uncertainty assessment is generally the responsibility of code writers as it can be challenging to undertake. The policy of ISO/TC 67/SC 7 in preparing its suite of offshore structural standards was to minimise the extent to which users would need to conduct such assessment. This was not because they were not necessarily capable of so doing but more because such undertaking is not well documented or otherwise standardised even amongst code writers so that appropriate guidance for users is not readily available. 'Uncertainty' is not presently defined in ISO 19900 [1]: it only appears 12 times in the document. However, its main superior standard, ISO 2394 [2], not only defines uncertainty (3 times) but also devotes an entire clause to its representation and modelling. Much of this is directly relevant to offshore structure uncertainty assessment but it is working at a simpler or more basic level than is commonly practiced on offshore structures. There is thus a need for ISO 19900 to supplement the information and guidance given in ISO 2394 with advice more directly relevant to offshore structure application, and with possible examples.

Uncertainty assessment is an essential ingredient in SRA (Structural Reliability Analysis). Uncertainty is normally defined in terms of a mean or bias value, a standard deviation or coefficient of variation (COV) and a distribution.

2 CRITIQUE OF ISO 2394

2.1 Introduction

For convenience, a summary of the contents of ISO 2394 Clause 6 "Uncertainty Representation and Modelling" is included here as Annex A.

2.2 Interpretation of Clause 6 for ISO 19900 application

Much of Clause 6 is directly relevant to ISO 19900 series of standards but offshore practice differs not insignificantly in a number of particular instances.

ISO 2394 treats environmental including seismic actions, if frequent, as variable actions with the obvious consequence that they both attract the same partial action factor. ISO 19900 treats environmental loading as a hazard although, surprisingly, does not treat gravity loading as a **hazard** despite the fact that gravity loading has "the potential to cause any, or all, of human injury, damage to the environment, and damage to property" [ISO 19900 definition 3.24] and is treated as a **design situation** [3.17] "for which the design demonstrates that relevant limit states are not exceeded".

A simple comparison is presented in Table 1 of ISO 2394 and ISO 19900 combined action factors when classing environmental loading as a variable action, as in ISO 2394, or as a separate class, as in ISO 19902 and using the default value of ISO 19902 environmental action factor. The design situations assume equal proportions of permanent and variable loading (G) combined with environmental loading (E) in the proportions listed in column 2 of the table.

Table 1. Comparison of ISO 2394 and ISO 19902 combined action factors.

Design situation	G:E	ISO 2394	ISO 19902
Operating condition	0.8:0.2	1.42	1.36
Extreme condition	0.2:0.8	1.10	1.30

In this simple example, the ISO 2394 extreme combination is unrealistic but the operating condition combination gives

some indication of the effect of using the ISO 2394 versus the ISO 19900 classification of environmental loading.

ISO 2394 classes infrequent earthquakes as accidents but not infrequent metocean actions, i.e. it has no abnormal action category, in contrast to ISO 19900.

The ISO 2394 Clause 6.4 treatment of model uncertainty seems unnecessarily complicated in that, for each basic variable, which in itself is characterised probabilistically, another variable is introduced at Equation (7) to account for the uncertainty which is also to be characterised probabilistically. Surely if the basic variable is correctly defined in statistical terms, which will account for any inherent uncertainty, there is no need a second variable to perform the same function. An inference here might be that the basic variable is no longer characterised probabilistically.

What Clause 6.4 does not do and, which is common practice in SRA, is to introduce a modelling uncertainty parameter (often designated X_m) which acts on the entire model usually independently of any of the basic variables. This X_m is then characterised probabilistically by comparison with appropriate data. Within such a SRA, some of the basic variables can still be treated as random but many are treated deterministically basic on the experience of findings of many such SRAs.

The evaluation of experimental results is described in ISO 2394 at Sub-clause 6.5 and Annex C while it is covered in ISO 19902 at A.7.7.2. For those not overly familiar with ISO 2394, they are likely to find Sub-clause C.5 *Direct evaluation of the test results* C.5.2 *Partial factor design* challenging. On the plus side, it is effectively a reproduction of the corresponding sub-clause in the 1998 edition of the standard but, in that, it was titled *Partial factor design: Bayesian method* and was preceded by *Partial factor design: Classical approach*. Deletion of the title extension *Bayesian method* is a loss because, without it, the ability to appreciate the contents of the current version is limited. Further, in adopting the earlier text, Example 1 and its Note that immediately follows Table C.1 in the current version, contain at least two errors as well as a misleading statement.

Table C.1 appears to be based on the non-central t-distribution but there is, unfortunately, no indication of the corresponding confidence level. It might be 75 % on the basis that this was the confidence level adopted in the 'sample size coefficient' tables in Sub-clause D.5.2 *Partial factor design: Classical approach* of ISO 2394:1998. There appears to be no indication as to why 75 % was chosen. This seems particularly unhelpful because ISO 19902 A.7.7.2 states that it uses the 50 % level based directly on that adopted in EN 1990:2002 [3] Table D1. However, given the Note at D7.1 (1) of EN 1990, the inherent confidence level appears to be 75 %.

3 LESSONS LEARNT FROM EXISTING STUDIES

3.1 Introduction

Extensive information is available on uncertainties from the many relevant works that have been conducted over the years. Of particular relevance are SRA studies because uncertainties are germane to their undertaking. Possibly of more importance are the *calibrations* [4] that have been performed to exploit past successful practice to derive partial load (action) and resistance factors for implementation of limit state-based offshore design practice. The significance of these works is that it is not just the researcher executing the work who defines the uncertainties for the loading and resistance variables but that they are also acceptable to the joint industry project steering committee responsible for overseeing the technical quality of the work.

Moan, in his keynote paper at the 2014 OSRC in Houston [5], discusses the risks and hazards affecting the limit states, safety and reliability of fixed and floating offshore structures and some of their important components, and how these can be affected by inspection, monitoring and maintenance. So ULS (Ultimate Limit State), FLS (Fatigue Limit State) and ALS (Accidental Limit State) are addressed in detail with respect to sources of loading, reliability measures and partial factors. Snell, in his keynote paper at the Houston OSRC [6], outlined past major incidents that would most probably have been averted if the design had been in accordance with the ISO 19900 series of standards and those that probably would not. The latter were system incidents where mechanical and control systems were critical to overall structural system performance.

Service Experience [5] recalls many of the major accidents that have marred the safety record of offshore structures as well as serious cracking particularly of semi-submersibles and ship-type structures. Stationkeeping system reliability, in particular, mooring line and DP failures, and their causes are considered. Safety Management addresses the challenging issues of gross errors, unknown phenomena, inadequate safety margins, robustness and the impact of human intervention in operations.

3.2 Fixed steel structures

Calibration studies for fixed steel structures include the works of Moses [7] in the initial development of API-RP 2A-LRFD [8], AME to extend API 2A-LRFD to the North Sea [9], Moses in a partial resistance factor determination [10] for a draft of ISO 19902 [11], and BOMEL [12] in a determination of a partial action factor for implementation of ISO 19902 to North Sea design practices.

Of course, one should not just adopt the values of the uncertainties contained in such documents without critically reviewing their relevance for the application in mind, especially the older works because of the availability of more up to date data and information. In this context, the informative annexes of some of the ISO 19900 series of standards can be useful because they identify the references and other sources of data exploited in their development and, in ISO 19902 in particular, present the statistical characteristics for tubular member modelling uncertainty parameters with respect to screened databases of test results especially in the case of tubular members – see Table 2 for a summary of the calculated uncertainties.

Table 2. ISO 19902 Tubular member uncertainties.

Loading condition	Bias	COV
Tension	-	-
Column buckling	1.057	0.041
Local buckling	1.065	0.068

Bending	1.109	0.085
Hydrostatic	1.142	0.124
Local buckling & bending	1.246	0.067
Column buckling & bending	1.030	0.082
Tension & pressure	1.075	0.098
Local buckling, bending & pressure	1.199	0.134
Column buckling, bending & pressure	1.197	0.091

Unfortunately, although similar information on tubular joints was contained in drafts of ISO 19902, it inexplicably did not appear in the final publication. In any case, with the revisions to tubular joint strength formulations currently underway for the second edition of ISO 19902 imitating the updates to the API RP 2A-WSD 21st Edition Errata and Supplement 2 formulations [13], the statistics would have had to have been revised to reflect the biases and COVs listed in Tables B.7.1 to B.7.3 of [13]. These tables cover K-, Y- and X-joints, respectively, for the usual four loading conditions of brace compression, tension, in-plane bending and out-of-plane bending. Separate modelling uncertainty parameter statistics are presented for physical test results and non-linear finite element analyses (FEA). From a perspective of future use of such results in a SRA, it perhaps would have been better to have combined the two.

In connection with metocean conditions, ISO 19901-1 provides quite limited information on uncertainties. It suggests that the accuracies achievable on significant wave height based on hindcasting is a mean error of 0.1 m, a COV of 10-15 % for storm peak, and a COV of some 20 % for all vales over a long continuous period. No other uncertainties are discussed.

Although environmental loading is discussed at some length in ISO 19902 A.9 as well as values for the associated partial action factors and even reliabilities (although it is planned to delete these from the forthcoming second edition), no attempt is made to summarise the corresponding modelling uncertainty parameter characteristics. Perhaps this is the relevant clause in which to report such values which could include the following:

- Heideman and Weaver [14] API global recipe for jackets using a wave by wave analysis – COV = 25 % (cited in [5])
- Digre, et al. [15] more realistic wave conditions than [14] – COV = 7 % (cited in [5])
- Efthymiou, et al. [16] typical storm maximum wave height – mean = mpm (most probable maximum), COV = 7.5 %, drag-dominated loading – mean = mpm, COV = 15 %, 20-year loading – NNS bias = 0.81, COV = 26.5 %, GoM bias = 0.79, COV = 32 %
- AME [9] 20-year loading NNS – bias = 0.877, COV = 18.8 %
- BOMEL [12] 20-year loading – bias = 0.852, COV = 21.6 %.

In assessing the applicability of the above statistics, one has to check carefully as to whether the environmental loading is determined on a wave by wave approach, relates to a single storm, or relates to a typical design return period of, say, 20 years. One-to-one comparisons are not always possible.

3.3 Floating steel structures

In one respect it is questionable whether there is a need to quantify uncertainties for floating steel structures given their design is no really the responsibility of ISO 19904-1 as their scantlings are normally determined in accordance with the RCS (Recognized Class Society) rules. However, ISO 19904—1 does specify minimum action and resistance factors for checking the design of components in general, and then of monohull hull girder bending and shear strengths in particular. Therefore, from a monohull system perspective at least, ISO 19904-1 does set the primary requirements so they need to be checked that they generate suitable probabilities of failure of the hull system.

ISSC (International Ship and Offshore Structures Congress) provides a valuable source of data and information on the uncertainties of ship structures and their components that readily applies to monohulls as well as to many of the components of semi-submersibles and spars. For example, the aim of the 1994 Congress Committee V.1 Applied Design – Strength Limit State Formulations [17] was to compare "formulations related to the ultimate buckling and the fatigue strength offshore structural components of ships, offshore structures, and other marine structures" with "available experimental data compiled in terms of probabilistic measures to evaluate the accuracy of these formulations for used in reliability-based design procedures". The determined uncertainties are listed in Table 3.

Table 3. Floating Structure Component Uncertainties [17].

Component	Bias	COV
Plates in compression		
◦ average imperfections	1.05	0.13
◦ actual imperfections	1.13	0.07
Stiffened plates in compression		
◦ BS 5400-3 1982	1.10	0.13
◦ DNV Buckling Notes 30.6 1987	1.10	0.16
◦ ECCS Column approach 1990	1.08	0.14
◦ ECCS Orthogonal approach 1990	1.14	0.15
◦ Imperial College methodology	1.10	0.08
Stiffened plates - compression and lateral pressure		
◦ DNV Buckling Notes 30.6 1987	≈1.22	-
◦ Imperial College methodology	≈1.15	-
Stiffened Girders – Bending		
◦ BS 5400-3 1982	0.99	0.04
◦ Cooper model	0.99	0.05
Stiffened Girders – Shear		
◦ Cardiff model	0.99	0.065
◦ BS 5400-3 1982	1.37	0.26
Tubular columns		
◦ API RP 2A-WSD 1991	1.14	0.07
◦ API RP 2A-LRFD 1993	1.02	0.08
◦ ECCS	1.0+0.1λ	0.05-0.16
Unstiffened and Ring stiffened cylinders		
➢ Local buckling		
◦ API 2U	1.07	0.08
◦ Cho & Frieze	1.05	0.14
➢ Bending		
◦ API LRFD	1.16	0.09
◦ Moan et al.	0.87	0.07
➢ Hoop buckling		
◦ API LRFD	1.12	0.11
◦ NPD	1.15	0.08
◦ ECCS	1.21	0.11
➢ Tension -hoop		
◦ API LRFD	1.20	0.16

◦ Merchant-Rankine	1.16	0.09
➢ Compression -hoop		
◦ API LRFD	1.15	0.10
◦ NPD	1.45	0.20
◦ NPD modified	1.03	0.08
➢ Bending -hoop		
◦ API LRFD	1.29	0.14
◦ API 2U	1.44	0.08

λ is non-dimensional slenderness

Reference [17] also considers orthogonally stiffened cylinders and tubular joints both ULS and FLS.

FPSOs:

The corresponding publication of the previous congress [18] undertook a SRA of a FPSO installed in the Oseberg area of the North Sea for which a reasonable amount of site-specific environmental data had been collected together with full scale measurements. Both linear and non-linear wave load analyses were performed from which non-linear correction factor to be quantified as sagging 1.15 and hogging 0.85 and a standard deviation of 0.03 to account for the spread of results. Short-term still water bending (SW) stresses closely followed Rayleigh. Non-linear vertical wave-induced maximum bending moments (WI) followed Gumbel distributions. SW and WI were simply added aided by an *ad-hoc* combination coefficient. The arrival rate for SW loading was 1 per day and for WI loading it was 6.3 s. Comparisons between measured stresses and linear wave load analyses yielded a bias of 0.932 and COV of 0.164.

SRA was performed on the strengths of a stiffened deck panel and two hull girder models and the fatigue of a deck detail. All the uncertainty modelling is given in detail in [18].

Subsequent ISSCs up to the 17th Congress continued to review floaters from design, fabrication, operation, full-scale response and reliability viewpoints, sometimes concentrating only on FPSOs, but more aimed at literature surveys than at conducting significant calculations. They continue to be a valuable repository of such information.

SEMI-SUBMERSIBLES

In the 1980s, a considerable number of RSR and SRA studies of semi-submersibles were undertaken as summarised in [19]. Undoubtedly much of this was spurred on by the tragic events of the Alexander L. Kielland [20] and Ocean Ranger [21] in 1980 and 1982, respectively. However, the causes of these were not so much related to the strength design of the structure and its components but more to human factors and the wider range of hazards associated with floating structure such as hydrostatic stability, damage stability, collisions, and capsizing/sinking. As a consequence of these events, rules and regulations pertaining to semi-submersibles in particular and floaters in general were tightened which significantly reduced their risk profile. One particular requirement introduced in Norway as a direct result of the first event was that "*the unit should withstand loss of buoyancy from either the whole or a major part of one column, but without any requirement to return to the upright position. The objective in this case was to allow the crew time to evacuate the unit*" [22].

The extent of this reduction in risk profile was captured in [23] where a review of the WOAD database revealed no structural design-related damage events throughout the 1990s. However, in terms of defining uncertainties for typical semi-submersible structural components, possible more arose from the SRA studies conducted as part of a joint industry project to develop code requirements for tension leg platforms (TLP) [24]. The results for some of the modelling uncertainty parameters for the strength formulations stiffened cylindrical components that emerged from this work are captured in Table 3. Some typical uncertainties related to this are listed in [25] and [26].

3.4 Other technologies and disciplines

The above-cited uncertainties have primarily addressed steel structures in non-arctic environments. ISO 19900 covers any material used offshore and, of course, soils. Many of the issues relevant to steel also apply to other materials as well as soils although the uncertainties associated with steel tend to be less 'uncertain' than for other materials and soils. The uncertainties associated with ice and related loads for arctic structures are discussed at length in the calibration study to derive actions factors for ISO 19906 [27].

4 GROSS ERRORS

As pointed out by Moan [5], the information collated above is well suited to exploitation by SRA. However, SRA does not normally deal with gross errors so cannot by itself be used to justify adequate safety. Some gross errors mentioned in [5] include:

- Fatigue failure of the Ranger 1 jack-up
- Fatigue failure of a brace of the Alexander L. Kielland
- Flooding through a broken window of the Ocean Ranger semi-submersible
- Accidental release of explosive gas on the P-36 semi-submersible.

Interestingly, ISO 2394 [2] does not refer to gross errors but, instead, requires robustness of a system to be established on the basis of risk assessment.

Moan [5] notes the usefulness of risk assessment in dealing with possible gross errors. However, he also notes that for structures comprised of a number of major subsystems such as floating structures, each subsystem is normally designed with respect to criteria particular to the subsystem and, consequently, trying to assess the acceptability or not of the complete system can be challenging. He notes that consideration of accidental limit states (ALS) in design goes some way to creating robust structures that will, as a consequence, help to reduce the consequences of gross errors but obviously not completely.

The complexity of highly advanced and integrated systems need special consideration. Structural standards primarily consider components and systems not dependent on the function of mechanical, electrical and numerically controlled systems. For integrated systems the design equations and procedures cannot normally be fully validated by physical or system testing prior to implementation. Floating structures in particular introduce levels of complexity that are a step up from classic structural systems. They also use components such as ballast and vent valves derived from shipping and drilling semi practice but without the option to dock them at intervals for maintenance.

The following examples are mentioned in [28]:

- In 2005 a large production semi-submersible platform was evacuated due to the anticipated close proximity of Hurricane Dennis. Personnel returned to find it with a 20° list. The two port columns had flooded due to a failure in the ballast system that would have been inconsequential but for the vessel being unmanned for a substantial period.
- In 2002 an FPSO listed to a near-critical angle, before a counter-flooding and pumping operation managed to right the vessel. An electrical failure caused the FPSO's 17-tank ballast system to malfunction.
- In 2005 and in preparation for Hurricane Rita, a TLP was evacuated. Following the passage of Hurricane Rita, it was found floating upside down, grounded. A probable cause of the capsizing was the loss of integrity of the tendon systems in one of the pontoons; specifically, at the bottom connector system.
- Some floating production platforms have lost their drilling packages during hurricanes principally due to inadequate tie down.
- Other failures have included an FPSO which irreparably damaged its hull due to overpressure under loading arising from an error in operations, mooring connector component failure due to manufacturing errors and out of plane bending fatigue causing failures of otherwise soundly manufactured mooring components in relatively benign conditions.
- Not experienced as a failure but of considerable consequence should adverse circumstances arise are FPSO designs that rely on thruster systems to maintain vessel heading under extreme conditions and vessels where it is operationally difficult to maintain adequate watertight sub-division whilst inspecting compartments.

Uncertainties in the safety of a system can be introduced due to a number of aspects: weak links in the system design, undetected poor quality assurance and testing during fabrication, inappropriate operational procedures, recommendations from risk assessments not carried through due to poor handover between design, construction and operation teams, etc. An example of possible mitigation measures for the specific case of a mooring system is given in [28].

In a more formalized manner [29] a comprehensive risk assessment should cover:

- Human Performance difficulties which covers human and organizational factors. There is valuable material from the review of high profile incidents [30], [31], [32] that specifically relates to this category.
- Equipment difficulties which covers defective parts, poor design, poor maintenance, etc. Here we are looking at reliability measures in general industrial applications based on more extensive test and failure data such as Mean Time Between Failures. These type of issues may be of interest to offshore systems when looking at uptime / downtime / SLS and failure of systems such as DP, TLP tendon mechanical components, etc. In this respect it has to be recognized that it may be impractical to fully physically test very large components and robustness in design is the primary safeguard.
- Use of numerically controlled systems normally improves day to day operational performance but the control system may not react appropriately to rare combinations of extreme circumstances. Should such an event happen operators may not be able to detect anomalies and respond quickly enough to avert major problems.
- Natural Disasters which are essentially the low probability events considered in ULS and ALS checks assuming a properly designed, maintained and operated structure.

When considering ULS and ALS checks it is penalizing to overdesign a given system to mitigate such errors and uncertainties. This may however be economically a more rational option than attempting to define all possible load combinations and undertake extensive physical testing. Designers may need to be discouraged from optimizing a system leaving little margin to cope with unexpected events. It is therefore important to more formally define robustness and how it relates to existing codes of practice. The Society of Naval Architects and Marine Engineers (SNAME) is currently working on a Technical and Research (T&R) document to better define robustness of floating production systems.

To facilitate robustness assessment, ISO 2394 Annex F introduces consequence classes, from 1 to 5. "*Smaller offshore facilities*" are in Class 4 with "*Major offshore facilities*" in Class 5. The analysis method for each is:

Consequence Class 4

Extensive study and analyses of scenarios leading to structural collapses utilizing risk screening meetings involving experts on all relevant subject matters. Detailed assessments shall be undertaken using dynamic and nonlinear structural analyses and risk analyses rigorously addressing direct and indirect consequences.

Consequence Class 5

As for Consequence Class 4 but with the addition of the involvement of an eternal expert/review panel for quality control.

Annex F goes on to provide guidance for cases where a risk and reliability based approach is not required and lists the following as correspondingly appropriate design methods:

- Event control – applicable when dealing with identified hazards
- Specific load resistance – given an accidental event, designing the local structure to absorb/resist it
- Alternative loads paths – a well-known offshore approach
- Consequence reduction.

Guidance is also given on how to execute a formalised risk assessment involving costs associated with direct and indirect consequences measured in either the number of casualties or monetary units, depending whether the emphasis is on life-safety or not.

A robustness index is suggested as one way to assess how one robustness scheme might compare with another involving the ratio of direct consequential risk to the total consequential risk which also includes the indirect consequential risk. The ratio takes values between zero and unity; the closer the ratio to unity, the more robust the scheme.

Alternative measures of robustness are also discussed. One exploits probability of failure determinations for the structure in the intact condition $P_{f(intact)}$ and in the damaged condition $P_{f(damaged)}$ through the redundancy index:

$$RI = [P_{f(damaged)} - P_{f(intact)}]/P_{f(intact)} \quad (1)$$

Values of this index can apparently range from zero to infinity with smaller values indicating larger robustness.

The last measure is incorrectly attributed to ISO 19902 [11]. It is called the Residual Influence Factor and is defined using the *RSR* in the form

$$RIF_i = RSR_{fail,i}/RSR_{intact} \quad (2)$$

in which RSR_{intact} is the *RSR* of the intact structure and $RSR_{fail,i}$ is the *RSR* of the structure with member *i* either failed or removed.

Some of these measures could prove useful for quantifying relative robustness when considering the effects of gross errors.

5 OUTLINE OF PROPOSED ISO 19900 ANNEX

The content on Section 3 above has served to indicate the extent of uncertainty data and information available in relation to just a selection of the offshore structures and their components. It clearly is quite extensive but, for application, needs to be up-to-date and readily available for those who seek to exploit it. As has been suggested in connection with the databases that have been used to quantify tubular member and tubular joint modelling uncertainty parameters, the use of the IOGP Interim Solution website as an appropriate repository seems to make sense. However, that does mean ownership is required with the obvious short-term solution being this Technical Panel but in the longer term, presumably it would have to be WG 1 itself.

The calculation of uncertainties varies from relatively simple approaches for quantifying those associated with, e.g., the strength of structural components, as reflected in the statistics currently quoted in ISO 19902, to technically-challenging techniques for determining those, e.g., associated with metocean conditions.

Uncertainties associated with foundations are important and difficult to deal with. Geotechnical design for ULS is generally aimed at maintaining foundations under limited deformations as they provide the boundary conditions for the structure. Foundation installation is based on failing the surrounding soil until strong enough material is reached to balance the installation loads and resist design storm loads as if such loads would occur immediately after installation.

ALS assessments may take advantage from the fact that soil properties may progressively recover strength with time and in addition structures with a degree of flexibility may accommodate larger foundation deformations without overstressing its members. The dynamic cyclic behaviour needs to be properly understood to avoid unconservative assumptions.

Uncertainties affecting foundation behaviour do not relate only to calculation methods but fundamentally depend on the borehole distance from the structure, borehole depth and overall quality of the site survey. Reference is made to ISO 19901-8: Marine soil investigations [33].

Notwithstanding, in each case, the approach would need to well defined so the resulting uncertainties are not unnecessarily increased as a result of human interference.

A possible outline for such an annex is presented here in Annex B.

6 CONSEQUENCES FOR DESIGN OF OFFSHORE STRUCTURES

It is difficult to simply assess the impact of formalising uncertainty modelling and assessment on offshore structure design given that for over 30 years, offshore structure code writers have been exploiting such information particularly when determining partial load/action and resistance factors. To encourage more widespread use of uncertainty data would mean a change of policy for SC 7. It would also require that internationally agreed procedures for executing SRA be put in place.

7 CONCLUSION

Sources of uncertainty for structural components of fixed and floating offshore structures are identified and some relevant values listed. The need to formalise procedures for quantifying uncertainties particularly for actions and geotechnical parameters is highlighted. Interaction between non-structural subsystems in the structural performance of particularly floating structures is highlighted as a major issue. Robustness is seen as one answer for mitigating the effects of the failures/mal-operation of non-structural subsystems.

A contents list for the proposed ISO 19900 annex on Uncertainty Assessment is presented.

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ANNEX A – CRITIQUE OF ISO 2394:2015 CLAUSE 6

A.1 Introduction

Clause 6 is comprised of six sub-clauses that deal in turn with:

1. General
2. Models for structural analysis
3. Models for consequences
4. Model uncertainty
5. Experimental models
6. Updating of probabilistic models.

Sub-clauses 1 and 2 provide considerable detail but 4 to 6 far less so. Salient features of each are summarised in the following subsections.

A.2 Sub-clause 6.1 General

There are two broad categories – aleatory or physical uncertainties and epistemic uncertainties which arise from inadequate information. A basic variable can have both types which can change through the life of a structure on the assumption that the design phase is part of this life. In SRA, there is no need to account for type of uncertainty – the argument is that differentiation is only important when seeking to reduce the uncertainty because the type will point the direction for efforts to achieve this.

Geotechnical uncertainties are particularly significant.

A Bayesian approach is preferred to the frequentist approach.

Basic variables can be represented as a random variable, a random process, or as a random field: these can be discrete as well as continuous. Dependency between random variables can and does exist.

An outcome is a sample of a population. Populations are probably best described in aleatory terms while a sample (outcome) is best described in epistemic terms. Characteristics of a population, to distinguish it from a sample, are:

- nature and origin of the random quantity
- spatial conditions, e.g., the geographical region considered. It is noted that in an international standard it can be necessary to divide a population into sub-populations on geographical grounds
- temporal conditions, e.g., design service life.

Some variable modelling has hierarchical dependencies representing different origins, times scales of fluctuation, or spatial scales. Wind is given as an example as it can be described in terms of a constant in time, a slowly fluctuating process in time, and a fast-fluctuating time process.

A.3 Sub-clause 6.2 Models for structural analysis

Structural systems address actions, geometrical properties, and material properties (or structural element properties). Reference is made to active and passive control measures – mainly the domain of risk analysis but can include dampers and similar.

Actions:

- to account for temporal (both short-term and long-term), spatial and directional features – it is noted that forces are direct actions that can be described as loads whereas indirect actions include imposed displacements or thermal effects

- action classifications are listed:
  - Permanent
  - Variable - ISO 2394 classifies environmental actions as variable actions
  - Accidental – small probability of occurrence and generally of short duration – it is noted that an accident action is not necessarily related to an accident
  - Earthquakes – either a variable or accidental action depending on their frequency.
- models need to represent these characteristics including interactions as necessary so normally a function of several variables.

Geometrical properties – differences from nominal values are normal and account for initial imperfections as necessary.

Material properties:

- to include all of the obvious values – strength, stiffness, angle of friction, etc., and more complex manifestations involving, e.g., multi-axial stress states, strain-rate effects, dependencies
- modelling to account for these plus differences between laboratory and prototype properties, random deviations between observed predicted values, effect of workmanship, time-dependencies, inspection uncertainties during and after fabrication
- classification – static response, time-dependent dynamic response, time-dependent degradation mechanisms – in each case, more details are provided.

A.4 Sub-clause 6.3 Models for consequences

Such models are primarily for risk-based scenarios but important for decision-making:

- loss of structural functionality, prior and post-failure behaviour, degree of primary & secondary damage.
- mitigating factors, human self-rescue actions, professional rescue actions,
- repair and rebuilding, environmental losses.

Use of fault and event tree analyses recommended, accounting for uncertainties.

Express consequences in numerical terms including fatalities and injuries, environmental damage and economic losses.

Fatalities need to be examined from a) numbers at risk, b) exposed and then injured or killed.

A.5 Sub-clause 6.4 Model uncertainty

Denotes the difference between structural analysis model results and actual behaviour.

The 'model uncertainties' are treated as independent random variables, one for each basic variable

The uncertainties need to be characterised by statistical properties, i.e., means and standard deviations.

A.6 Sub-clause 6.5 Experimental models

Recommended where no adequate calculation model exists or existing models are considered too conservative.

It is recommended that "The setup and evaluation of the tests should be performed in such a way that the structure, as designed, has at least the same reliability with respect to all relevant limit states and load conditions as structures designed on the basis of calculation models only."

Conditions not met during the tests such as long-term effects should be accounted for separately.

Relevant basic variables should be measured directly or indirectly for every test.

Test results to be evaluated statistically and should lead to a probability distribution. With this information, apparently design values and partial factors can be derived, as described in Annex C.

Annex C is titled "Design based on observations and experimental models". Firstly, it gives more details on some of the above items. It also gives a process for the direct determination of the design value from test results using a procedure that accounts for the number of tests and the impact this has on the outcome via the student distribution. A very similar procedure is given in ISO 19902 Clause A.7.7.2 although there appears to be at least one term that differs and ISO 2394 does not specify the exceedance level nor the confidence level, both of which are itemised in ISO 19902 models.

A.7 Sub-clause 6.6 Updating of probabilistic models

Recommended in cases where relatively high uncertainties exist for actions, structural properties and/or models, to achieve a more economical design. It can be based on quality assurance procedures during and after construction as well as lifetime inspection and monitoring planning. Observations can lead to posterior distribution for the random variables. Inspection uncertainties should be accounted for.

ANNEX B – PROPOSED CONTENTS LIST FOR ISO 19900 ANNEX ON UNCERTAINTY ASSESSMENT

B.1 Introduction

To make use of ISO 2394 definitions and clauses addressing uncertainty modelling but modified as described above for the offshore industry.

B.2 Uncertainty data

Before being entered into a database, experimental data, full-scale measurements, etc. needs to be subject to a quality control (QC) check. For tubular members and tubular joints, in the development of ISO 19902, this screening to remove results that were not necessarily consistent with good practice – see A.13.1, A.13.2, A.14.2.1,

A. Material properties
   i. steel
      a. yield strength and tensile strength
      b. Charpy impact tests
      c. CTOD
      d. CE and modified CE
      e. SCFs
      f. Fatigue tests and S-N curve, endurance limits
      g. …
   ii. concrete
   iii. soils
      a. cohesive - undrained shear strength

b. cohesionless – skin friction factor, end bearing factor, limiting values,
c.
iv. synthetic fibre
v. piles
a. static strength
b. cyclic behaviour
c. lateral loading
vi. cohesionless
vii. …

B. Component strength
i. tubular members and joints
ii. other components
iii. plates and stiffened panels
iv. stiffened cylinders
v. chains, tendons, cables, ropes, …
vi. …

C. Component fatigue
i. tubular joints
ii. other components
iii. ship semi-submersible and spar details
iv. chains, tendons, cables, ropes, …
v.

D. Metocean
i. hindcast data – necessarily regionally-based - following interpretation to give short-term and long-term wave height distributions and heights in an interval T based, e.g. on Tromans and Vanderschuren methodology (TVM)
ii. measured data
iii. …

E. Responses
i. measured
ii. calculated and calibrated against measured values – studies on particular structures
- studies on generic structures such as the TVM pile
iii. …

F.

G. ….

B.3 Uncertainty calculation techniques & procedures

Describe the techniques and procedures to be adopted when determining the properties, strengths, metocean parameters, etc. They need to be clear and unambiguous to minimise misinterpretation.

A. Material properties
i. steel
a. yield strength and tensile strength
Yield and tensile strength shall be determined in accordance with standardised procedures. Ideally the strain rate shall also be recorded
b. Charpy impact tests – see ISO 19902 20.2.2.4
c. CE and modified CE – see ISO 19902 20.2.2.4.2
d. CTOD – See ISO 19902 Annex B
e. SCFs
f. Fatigue tests and S-N curve, endurance limits
g. …
ii. concrete
iii. soils
a. cohesive - undrained shear strength
b. cohesionless – skin friction factor, end bearing factor, limiting values,
c.
iv. synthetic fibre
v. piles
a. static strength
b. cyclic behaviour
c. lateral loading
vi. cohesionless
vii. …

B. Component strength
i. Screened databases – Experimental results
ii. chains, tendons, cables, ropes, ..
iii. …

C. Component fatigue
i. tubular joints
ii. other components
iii. ship semi-submersible and spar details
iv. chains, tendons, cables, ropes, ..
v.

D. Metocean
i. hindcast data – describe
ii. measured data

B.4 Example calculations

A. Component strengths
Describe the various sources of tubular member strength data and why particular test series were deleted from the database.

B. Maximum wave heights
Describe application of TVM to hindcast data for determination of maximum wave heights in an interval T.

17.4 Presentation by Simen Moxnes

NTNU Norwegian University of Science and Technology

International Association of Oil & Gas Producers

Special Session:
Development of the Third Edition of ISO 19900

SC 7/WG 1/TP 3 Uncertainty Assessment

Paul Frieze, PAFA Consulting Engineers

The 3rd Offshore Structural Reliability Conference
OSRC2016
14-16 September, Stavanger, Norway

TP 3 Scope & Membership

- Singapore January 2016 WG 1 project
- Technical Panel 3 tasks
  - New annex on uncertainty assessment
  - Impact on any normative clauses in the main Standard
- Members - appointed
  - Paul Frieze, PAFA Consulting Engineers (lead)
  - Sathish Balasubramanian, ExxonMobil
  - Torgeir Moan, NTNU
  - Marc Maes, Aleatec AS Inc.
- Members – co-opted/assigned
  - Alberto Morandi, American Global Maritime
  - Richard Snell, Oxford University

TP 3 Uncertainty Assessment - Activities

- Reviewed the requirements of ISO 2394:2015, *General principles on reliability for structures* with particular emphasis on application to the proposed ISO 19900 Annex on *Uncertainty Assessment*
- Proposed a Contents List for the Annex
- Sought additional members for the TP
  - Alberto Morandi
  - Richard Snell
- Participated in WG 1 telecons
- Prepared the OSRC paper

TP 3 Activity – Review of ISO 2394

- ISO 2394:2015, *General principles on reliability for structures*
  - Has 3 different definitions relating to *uncertainty*
  - Entire clause (6) devoted to *Uncertainty representation and modelling*
    - Clauses 6.1 *General* and 6.2 *Models for structural analysis* – comprehensive
    - Clauses 6.4 to 6.6 – relatively brief
- In contrast, ISO 19900 has no definition of *uncertainty* and the term only appears 12 times
- Many ISO 2394 provisions on *uncertainty* are directly relevant to ISO 19900 but, given the extensive use of SRA in the offshore industry, ISO 2394 seems to work at a simpler level than is usual in offshore applications

TP 3 Activity – Review of ISO 2394

Highlights:

- Treats environmental and seismic actions as variable actions although an infrequent earthquake is an accident but not infrequent metocean conditions, i.e. there is no such thing as an abnormal action
- The *calibration* procedure for deriving partial factors does not align with that used in the API/Moses work involving existing exemplary structures
- It seems to believe that the representative value of steel yield strength is a characteristic value (5%) whereas it is always a nominal code value
- 6.4 *Model uncertainty*, at first reading, seems to be describing *modelling uncertainty parameter*, a statistical parameter used in SRA studies to define the accuracy of a codified strength equation (model). However, it actually appears to be suggesting that each basic variable in the strength equation has its own modelling uncertainty parameter which is quite different from the one normally used in SRA studies.

TP 3 Activity – Review of ISO 2394

Highlights:

- Annex D (informative) is devoted to *Reliability of geotechnical structures*
- Provides guidance for the application of the normative requirements of ISO 2394 to geotechnical design using either Clause 8 *Reliability-based decision making* or Clause 9 *Semi-probabilistic method* which includes the *Partial factor method* (as adopted in the ISO 19900 Series of Standards)
- Good introduction to the full range of issues needed to be addressed when trying to quantify uncertainties in a both spatially and temporally varying medium
- When simplified a little, is directly amenable to application to steel and concrete structures and, to some extent, overcomes the above criticism concerning application of ISO 2394 to offshore structures
- Needs serious editing to define all the parameters used and to achieve consistency with the normative text

TP 3 Activity – What is "uncertainty"

Highlights:

- ISO 2394 does not make clear what is meant by uncertainties - example

  9 *Semi-probabilistic method* 9.2 *Basic principles*

  Introduces resistance R using the following general model

  $R = b\ \theta R(X, a)$ (17)

  X are the material properties

  a are the geometric parameters

  θ is the model uncertainty related to the resistance model

  b is the bias in the resistance model.
- Partly clarified at D.4 *Statistical chacracterization of model factors*

  Model factor M defined as ratio of "*measured to the calculated capacity*"

  $M = Q_m/Q_p$ (D.1)

  Q_m is measured capacity

  Q_p is predicated capacity

  Active SRA users would immediately recognise this as "modelling uncertainty parameter" and know that it is defined in terms of a bias, a standard deviation or COV, and a statistical distribution. Reference to "*statistical assessment*" in the following paragraph would suggest this is the intended scope of the term "uncertainty"

TP 3 Activity – What is "uncertainty"

Defining "uncertainty"

- The ISO 2394 definition of "uncertainty" is ambiguous.
- A layman's interpretation would probably be limited to "standard deviation" or "COV", i.e. no bias and certainly no associated distribution
- Personally I am not keen to redefine terms that have a relatively clear "lay" meaning with an ISO standard definition. A particularly good example is the term "risk" because without very strict editing of standards, the lay use appears not infrequently

Uncertainty is meant to encompass bias, standard deviation and probability distribution

- In the context of ISO 2394, "statistical modelling" seems a more appropriate phrase to describe the determination of probabilistic properties of bias (or mean), standard deviation (or COV) and distribution
- The statistical modelling is to covers basic variables as well as modelling uncertainty parameters (or modelling factor according to ISO 2394 Annex D) although in its general sense, variable could also include modelling uncertainty parameter
- Instead of ***Uncertainty modelling***, suggest ***Variable statistical modelling***

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TP 3 Contents List for Annex

1. Introduction

- Make maximum use of ISO 2394 but make clear where it does not apply because of preferred/established techniques used in offshore SRA studies

2. Uncertainty/Variable statistical data

- Database of uncertainty data - to be seen as a repository of (standardised) uncertainty data
- Ensure adequate QC procedures are in place and are applied to data before entry into database. Identify some appropriate procedures, e.g. database screening approaches used in creating ISO 19902 tubular member and tubular joint databases

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TP 3 Contents List for Annex

2. A. Material properties
   - i. Steel
     - a. Yield strength and tensile strength
     - b. Charpy impact tests
     - c. CTOD
     - d. CE and modified CE
     - e. SCFs
     - f. Fatigue tests and S-N curve, endurance limits
     - g. ...
   - ii. Concrete
   - iii. Soils
     - a. Cohesive - undrained shear strength
     - b. Cohesionless – skin friction factor, end bearing factor, limiting values
     - c. ….
   - iv. Synthetic fibre
   - v. …

   B. Component strengths
   - i. Tubular members and joints
   - ii. Other components
   - iii. Piles
   - iv. plates and stiffened panels
   - v. stiffened cylinders
   - vi. chains, tendons, cables, ropes, ...
   - vii. ...

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TP 3 Contents List for Annex

2. C. Component fatigue
   - i. Tubular joints
   - ii. Other components
   - iii. Ship, semi-submersible & spar details
   - iv. chains, tendons, cables, ropes
   - v. …

   D. Metocean
   - i. hindcast data – regionally-based - short-term and long-term wave height distributions and heights in an interval T based, e.g. on Tromans and Vanderschuren methodology (TVM)
   - ii. measured data
   - iii. ...

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TP 3 Contents List for Annex

3. Uncertainty calculation techniques & procedures

- Describe the techniques and procedures to be adopted when determining the properties, strengths, metocean parameters, etc. They need to be clear and unambiguous to minimise misinterpretation.

A. Material properties
- i. Steel
  - a. Yield strength and tensile strength – determined in accordance with standardised procedures. Record strain rate
  - b. Charpy impact tests - see ISO 19902 20.2.2.4
  - c. CTOD - See ISO 19902 Annex B
  - d. CE and modified CE – see ISO 19902 20.2.2.4.2
  - e. SCFs
  - f. Fatigue tests and S-N curve, endurance limits
  - g. ...
- ii. Concrete
- iii. Soils
  - a. Cohesive - undrained shear strength
  - b. Cohesionless – skin friction factor, end bearing factor, limiting values
  - c. ….
- iv. Synthetic fibre
- v. …

B. Component strength
- i. Screened databases – Experimental results
- ii. chains, tendons, cables, ropes, ..
- iii. …

TP 3 Contents List for Annex

4. Example calculations

A. Component strengths

Describe the various sources of tubular member strength data and why particular test series were deleted from the database.

B. Maximum wave heights

Describe application of TVM to hindcast data for determination of maximum wave heights in an interval T.

TP 3 Contents List for Annex

5. Gross errors, Human factors, Equipment performance

These topics represent those not covered by SRA or directly by structural design standards. However, as a source of hazards, they can be more important than structure especially for floating platforms.

ISO 2394 addresses some of this through its robustness requirements, based on risk assessment. It introduces a robustness index and other robustness measures. One of these, defined as *Residual Influence Factor* (wrongly attributed to ISO 19902) exploits RSR via

$RIF_i = RSR_{fail,i}/RSR_{intact}$

RSR_{intact} is *RSR* of intact structure

$RSR_{fail,i}$ is *RSR* with member *i* either failed or removed

At least one major oil company has had a process for many years to derive RIF_i for a fixed steel structure by automating USFOS to remove every member/joint in turn – an overnight exercise which provides the complete list next morning

TP 3 Impact on Normative Requirements

- For ISO 19900, direct impact of *variable statistical modelling* is likely to be minimum other than to improve consistency of terminology – similarly for other ISO 19900 series standards
- However, given the need, particularly for floaters, to ensure robustness, implementing even a modified version of the ISO 2394 requirements is likely impact on ISO 19900 normative requirements
- Because *variable statistical modelling* data is mostly used in SRA studies, this will help generate more consistency when such studies are performed. However, without a documented approach for executing SRA, real consistency will be hard to achieve
- Generally, it is likely to lead to more consistent application of procedures such as TVM because these are fully not documented as yet

Chapter 18

Discussions

The 3rd Offshore Structural Reliability Conference (OSRC2016)

14-16 September, Stavanger, Norway

Discussions

Edited by Torgeir Moan and Amir R. Nejad

Session 1: Metocean Conditions

Co-chairs: P. Tromans and O.T. Gudmestad

P-S1-1: Waves and Associated Current – Experiences from a Five Year Measurement Campaign in the northern North Sea (see also associated article: A-S1-1)
K. Bruserud, Statoil and Sverre Haver, UIS/NTNU, Norway . Presented by K.Bruserud

P-S1-2: Airgap and Safety: Metocean Induced Uncertainties Affecting Airgap Assessments (see also associated article: A-S1-2)
S. Haver, UiS/NTNU, Norway

P-S1-3: Wave Kinematics and Hydrodynamic Loads on the Tyra Jacket Inferred from Systematic Model Testing and Field Measurements (see also associated article: A-S1-3)
J. Tychsen, Maersk Oil, Denmark; M. Dixen, DHI, Denmark. Presented by J.Tychsen

Discussions:

Q by A van der Stap to J. Tychsen (JT): When we run structural analysis software, does the software not indicate the validity of the wave model for the water depth and wave steepness that we select? In view of this, is Stokes wave theory valid for a water depth of only 45 m?
A by JT: None of the standard models, especially those for regular waves, produce water surfaces or kinematics anywhere close to the breaking waves that actually occur. Breaking waves are highly transient and cannot be modelled by "equivalent" regular (or linear focussed) waves. The challenge is that no closed form solutions exist to fully non-linear irregular waves.

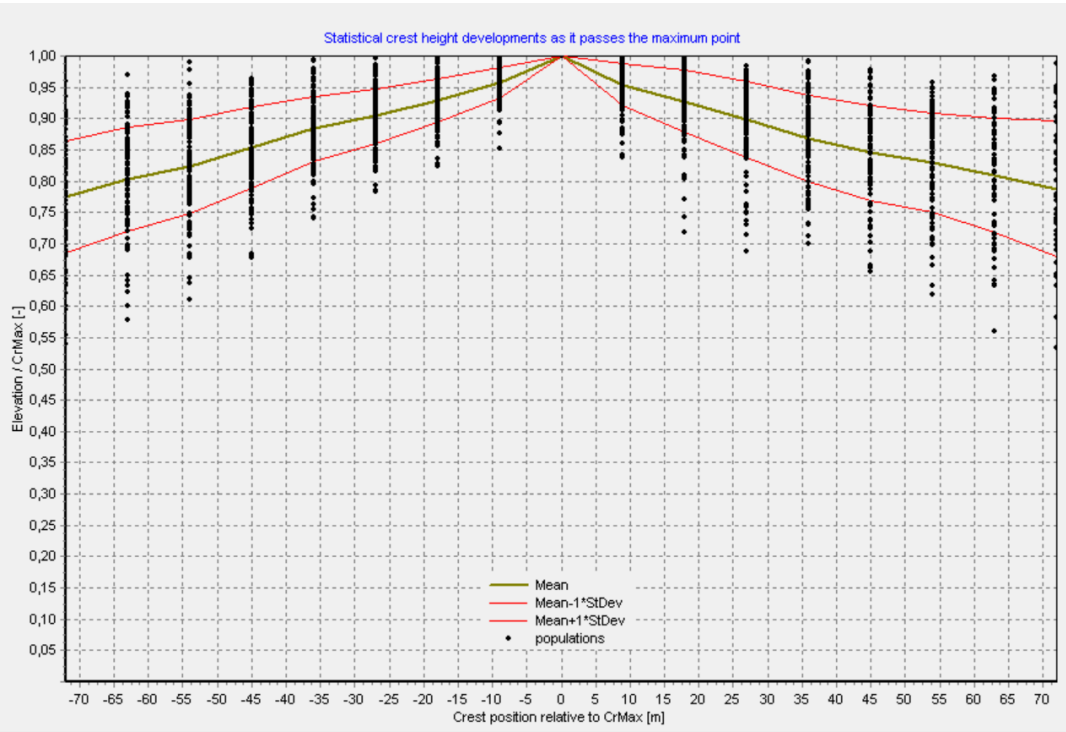
Q O. T. Gudmestad to JT: It was said that that every 2. or 3. waves are breaking. Could this be due to a bottom with a slope?
A by JT: All Tyra wave basin tests (sea states) are performed with a flat bottom in the basin. Actually, it may be the other way around (more events are breaking), as the target area is approx. 500m (full-scale) from the wave maker we might miss some non-linearities caused by interacting effects in wave groups travelling longer distances. Furthermore, it is noted, as highlighted in the presentation, that the breaking probabilities given are based on video inspections (≈7000 extreme events) and breaking is defined by the event breaks (top-spilling/spilling/plunging) within the field of view of the camera, i.e. a reasonably large reference area. It is important to note that these irregular waves are highly transient and for this reason will not sweep the entire reference area as an equivalent regular wave will do. It follows the breaking probability will be a function of the size of the reference area and is not a constant.

Q O. T. Gudmestad to JT : How are the findings from the studies implemented?
A by JT: The tests are used to calibrate a semi-empirical non-linear sea state model. This model is implemented in a Monte Carlo based simulation model and used to estimate the collapse probability for all main failure mechanisms in the Tyra structures. Further details on this work follows in a presentation in, Session 3.

S. Haver expressed the following opinion after JT's talk:

1. I agree that 5th order Stokes wave theory will underestimate wave crest in particular in deep water but possibly also for waves in shallow water as they approach breaking.
2. Wave breaking is affected by water depth. In deeper water (say above 80 m), expectation of breaking waves is low. The 5th order Stokes design wave approach is expected to on the conservative side for loads on structures in the deeper water (above 80 m).
3. The extensive study of JT is for a depth of about 40m. Further work - numerical analyses and model tests - are needed before make conclusions given in the JT's presentation (which I do not question) as being valid regarding the importance of breaking waves for deeper waters. At a larger depth e.g. Ekofisk waves breaking but less often. But further work on this for larger depths is very welcome.

Q by S. Haver to JT: What is the length of the crests of the extreme waves? If it is 30 m long or more, it is of same size as a platform, and may imply large loads – in particular if the waves hit the deck.
A by JT: The crest length is (as many other sea state geometric parameters) a highly stochastic variable. Using the 9x9 wave gauge array we can picture the variation. During the project we developed an application "ePlot" which presents the data of the largest events in each test. Below is a plot of normalised crest height (CR>13m) along the wave crest (normal to mean wave direction) in a 86h long test with Hs=11.5m; Tp=15sec. Sp=21. A significant variation is seen but on average a large crest (>13m) in this sea state will be approx. 25m long before the along crest decay is larger than 5% relative to crest peak. It is also seen that approx. 20% of the crests will be 50m long for the same 5% decay criteria. This is based on the assumption on wave symmetry which may not be fully correct but on the other hand we also "cut" the profile perpendicular to the mean wave direction which will lead to a too large decay for events not fully aligned with the mean wave direction.



Q by SH to JT: How far from uni-directional or how short-crested are the extreme waves in your studies?

A by JT: We tested a large matrix of Hs, Tp and Sp combinations for very long durations. For Sp we targeted a low spread (SP=50-60), a medium spread (Sp=20) and a high spread (Sp=5-8) variation. Looking at the most extreme crests it is clear that subjectively as well as indicative from force measurements, the highly non-linear extreme crests are more unidirectional ("2D") than the "general" directionality of the sea state in which they occur.

Reflections of a co-chair (Peter Tromans):

In the session, there were three very impressive talks, each presenting new and interesting ideas. They were particularly interesting for me as they touched on many areas where I have worked.

The first paper by Kjersti Bruserud and Sverre Haver visited the old question of what current to associate with extreme design waves. Chris Shaw asked me to look at some measured wave and current data about 1990 to try to develop some ideas on estimating an associated current. The data were from the well mixed water column of the UK sector of the North Sea and the currents were not complicated by inertial oscillations. Though the quality of the waves and currents left far more to be desired than those of Kjersti, we managed to develop a few ideas that evolved into Shell's LSM software for response-based design. One of those ideas was to use a simple structure model to identify to find the balance between wave and current. Hindcasts arrived and I never looked at measured joint data again. It is very pleasing to see Kjersti and Sverre doing it so carefully and using a response-based principle.

In the second paper, Sverre Haver gave the most eloquent discussion of air-gap and safety. In trying to put reliability analysis into practice in the 1990s, several of us in Shell learnt that almost all fixed structures would experience wave-in-deck before collapse under extreme storm load. After that, we pushed for higher deck elevations in new design, such that the return period of wave-in-deck should be comparable with the target reliability of the platform. This, for better or worse, decouples air-gap from the structural ULS and ALS conditions. We also became aware of the necessity of model testing to set this level for large volume structures or, indeed, as Sverre shows to be more practical, not setting it, but designing to withstand some deck impacts. Sverre also demonstrates that there is a lot of devil in the detail: for example, changing the wave spectrum influences the crest elevation a little and the wave inundation of the deck by quite a lot.

Session 2: Wave Environment and Loads

Co-chairs: C. Swan; S. Haver

P-S2-1: Demonstrating Draugen ALS/ULS Compliance despite Significant Wave/Ringing Load Increases since Original Design
G. Kuiper, Norske Shell, Norway

P-S2-2: Slamming Loads from Steep and Breaking Waves
G. Lian and T. Vestbøstad, Statoil, Norway

Discussions:

Question from G.Kuiper to the audience:
Q: Both ULS and ALS needs to be fulfilled. If only one of them should be fulfilled, which one would be preferred? (Could be special considerations related to concrete structures inherent in this question.)

Q from O.T. Gudmestad to G. Kuiper: The study was related to extension within the license period, will have to analyse the platform for use beyond the license period?
A: Further extension will have to be assessed..

Q from P. Tromans to G.Kuiper: In contour plots, wind driven sea and swell are both included. Could it be possible to separate Wind Sea and swell, generate the distribution for each of them, and then combine the distributions?
A by S. Haver: It would be difficult to separate the two wave systems in order to create the distributions.

Reflections of a co-chair S. Haver:
In the second session, two interesting papers were presented. Both presenters stayed very well on their time schedule, making the co-chairs job easy.

The first paper, presented by Guido Kuiper, A/S Norske Shell, presented the work done in order to demonstrate NORSOK compliance for Draugen in spite of a considerable increase in the metocean loading. In the original work, short term variability was not accounted for. Present versions of NORSOK standards require this explicitly to be accounted for. According to the presentation, this resulted in an increase in characteristic wave loads of about 30%. After an extensive model test in long crested seas at Marintek in Trondheim, compliance with the Accidental Limit State (10000-year check) requirement was demonstrated. However, after extensive further work full long term analyses , model tests in short crested sea at Imperial College and some assessments of GBS capacity, ULS compliance was reached.

Interesting remarks made by Kuiper during his presentations were:

- It is important to introduce short crested sea. It is less important how short crested the waves are considered to be.
- What is defining the acceptable safety of the structure in case of life extension: Is it ALS or ULS?

The second paper presented by Gunnar Lian did also deal with model testing and - in particular – with assessments of impact loads from breaking waves. The paper pointed out that the correct approach – in

principle - would be to do a full long term analysis which is presently out of reach if results are to be based on model test experiments. In practise, most test programmes will focus on testing some ULS and ALS sea states using the 3-hour maximum impact load as the primary variable. The challenge is then to select percentile of the 3-hour extreme value distribution in order to use the value as a ULS or ALS characteristic load. The paper did not answer this question, but the sensitivity to choice of percentile is demonstrated. The characteristic impact pressure is more or less doubled by selected the 99% value instead of 90% value. The paper gave an overview of results obtained for the Cat D semi-submersible with still water airgap of 16.6m in ULS conditions. A normalized estimate of the 90% value is presented over a considerable part of the front of the deck box of the semi. The examples included did clearly demonstrate that 16.6m is too small if major deck box impacts shall be avoided. The results can be taken as indicative, regarding characteristic ULS events, the adoption of 90% value is non-conservative, at least for the sea states located on the ULS metocean contour.

Session 3: Reliability of Jackets in Severe Wave Conditions

Co-chairs: M. Birades; J. Waegter

P-S3-1: Summary of the Impact on Reliability by the Tyra Field Extreme Wave Study 2013-15 (see also associated article: A-S3-1)
J. Tychsen, Maersk Oil et al., Denmark. Presented by J.Tychsen

P-S3-2: The Loads JIP: The Loading and Reliability of Fixed Steel Structures in Extreme Seas (see also associated article: A-S3-2)
C. Swan, M. Latheef and L. Ma, Imperial College, UK. Presented by C. Swan.

Discussions:

Q from H. Singh to J. Tychsen: Question about the applicability of the findings of the Tyra Field on piles in leg-pile platforms, considering that the structures of the Tyra field have skirt piles.
A by JT: Most of the Tyra platforms have a combination of leg and skirt piles. However, as part of platform strengthening leg piles have been grouted to increase the strength of the leg joints. Level shear conclusions for these structures are relevant as the jacket force flow is effectively equal to a cantilevered beam. In ungrouted (welded at the top of the leg) leg-pile platforms the force flow is different as the axial force in the pile can only transfer at top of leg. This change in force flow will in a given case need to be considered in the response model applied.

Q from A. Morandi to C. Swan: Question about the effect on wave in deck: Why not increase the freeboard in order to avoid wave in deck loads?:
A by C. Swan: The better solution would be to avoid wave in deck. However, existing structures may be exposed to WID. Design should include effects beyond 2. order (Forristal crest distribution?), otherwise we may be on thin ice. This is a complex issue. However, an increased airgap can be more easily addressed for new designs.

Q from M. Abraham to C. Swan: Considering the results of the Loads JIP; is it now needed to change the recipes for design, and are the conclusions region dependent, specific to the North Sea or similar areas?
A by C.Swan: If there is no wave in deck, existing recipes are conservative.
In case of wave in deck, conclusions are region dependent, further, the key issue is where failure occurs in the structure (especially in relation to elevation).

Notes by co-chair Michel Birades (in writing):
Linked to the JIPs results, the draft of a revised Norsok N003 standard already proposes a simplified approach introducing a correction factor to account for higher order wave effects and the area effect in lieu of a more refined assessment.. This proposal is still under discussion within the Oil Industry. Furthermore, after the OSCR 2016, the IOGP Offshore Structure Subcommittee has agreed to establish a Task Force to look into and advise the OSSC within one year on "What to do in face of the JIPs findings and how does this relate to the structural standards we have?"

Q from P. Smedley to C. Swan: Loads JIP concerns fixed steel structures. What about floating structures, semi-subs etc.? Will the large volume complicate the analysis?

A by C. Swan: To some extent, the findings for fixed platforms are transferable to floating structures, in particular for slamming and the air gap issues would also be transferable to semi-subs. But roughly speaking, CS thought that the non-linear effect of wave loads on floating structures, in general, could expect to be more critical : Interaction between steep waves and floaters will increase the loading. Examples: For FPSOs: Slamming. For GBS: Wave in deck.

Editor remark (T. Moan): An important additional aspect for floating platforms is to account for the motions, including nonlinear effects, diffractaion etc, to estimate the possible wave in deck phenomena is challenging. The guidelines emerging to check airgap for e.g. semi-submersibles are based on a linear response analysis and account for nonlinear effects on the the relative motions by using a correction factor (asymmetry factor) on the wave elevation.

Q from Y.S. Choo to C. Swan: Question about blockage. It was stated that recent research shows that the blockage factor used in current wave load recipes leads to an over-prediction of hydrodynamic loads. The question therefore concerned the possibility for wave energy to dissipate within the jacket, particularly with a significant number of conductor pipes, and therefore with the potential final effect to limit the wave height with respect to wave impact on the deck.
A by C. Swan: The answer was not fully in line with the question: "Jacket structures are not dense enough to create wave increase. But impact on risers and conductor pipes has to be studied"

Q from G. Kuiper to C. Swan : Question about dynamic amplification and how the dynamic effects of the waves are accounted for when pushover analyses are performed for jackets,?
A by C. Swan: Local wave steepness can create large dynamic amplification factors. Both Chris Swan and Jesper Tychsen mentioned that actual values of 1.6 and higher had been found in some of their calculations: Dynamiv analyses were dynamic analysis, and hence accounting for the dynamic wave load effects

Session 4: Reliability Based Calibration of ULS Code Criteria

Co-chairs: H.O. Madsen; A. Mangiavacchi

P-S4-1: Risk-Based Codification of Structural Design and Assessment: Benefits and Challenges
M. Maes, University of Calgary, Canada

P-S4-2: Uncertainty Assessment of Geotechnical Design and Calibration of Resistance Factors for Offshore Piles
F. Nadim, Presented by F. Nadim

Discussions:

Q from P. Frieze to F. Nadim: Could you please explain the thinking behind calculation of modelling uncertainty parameter in which the 'high' ratios of test strength predicted strength were 'deleted'. Having done a similar screening exercise in relation to tubular members for the first edition of ISO 19902, clear guidelines are necessary when conducting such screening exercises.

A. by F. Nadim: In reliability calculations, it is important to fit the probability distribution models for load and resistance to be representative for what is happening close to the design point. This means that we should focus on the situations where our resistance model overpredicts the capacity, and the actual capacity is lower than the predicted one, because this is the situation that could lead to failure. Therefore, in the assessment of the modelling uncertainty for an empirical pile capacity prediction method, we are only interested in the cases where the ratio of measured capacity to calculated capacity is less than 1. The cases for which this ratio is greater than one are not relevant for failure conditions.

Session 5: Accidental Collapse Limit State

Co-chairs: Y.S. Choo; T. Sildnes

P-S5-1: Standards – Targets and not Risk Management
D. Wisch et al., Chevron, USA. Presented by M. Abraham

P-S5-2: Assessment of Ship Collision Risk in the North Sea: Recent Guidelines (see also associated article: A-S5-2)
T. Moan, NTNU, J. Amdahl, NTNU, G. Ersdal, PSA. Presented by G. Ersdal and J. Amdahl

P-S5-3: Non linear, Dynamic Analysis of Long term Blast Loading on a Topsides Compression Skid
H. Singh, Shell Global Solutions, the Netherlands

Discussions:

Discussion related to P-S5-1:

C. by T. Moan (TM) : Human errors are important issues that need to be considered in the safety assessment. While avoiding gross errors is ensured by competent personnel doing the job, QA/QC, use of ALS criteria to ensure robustness is also a risk reduction measure. Human errors are not normally accounted for in the Structural Reliability Analysis (SRA) and when referring to target values it should be made clear whether e.g. failure probabilities refer to SRA or actuarial values.

A by M. Abraham: Agrees with TM. But, a trend, in the real world, is that many engineers tend to play a "computer game". It is easy to do analysis and get results. The challenge is how to control the quality of the analysis, how to correctly understand and interpret the analysis results, is the proposed analysing approach sufficiently realistic and reliable?

C by A. van der Stap: A modified crane collapsed. FE-analysis showed large stresses, it was explained with computational singularities. The certifying agency also missed this problem and approved the crane. Hence, it is important not to become analysts, but remain engineers.

Q from T.Moan to M.Abraham: In your talk on behalf of Dave you mentioned two aspects "in the way forward"; namely a practical design guidance in standards and at the same time a general framework. Practical guidelines tend to be quite specific while I interpret general framework to be a functional and not specific approach. Could you comment on this apparent contradiction?

A by M.Abraham/D.Wish: Standards are tending to be more equation and target centric with the focus becoming more analysis driven than design driven. Many historic provisions in codes relied on rules of thumb and/or a parametric basis. These historic provisions provided for a range of design conditions often not considered explicitly. With today's more analytic driven practices, critical design cases may be missed and not explicitly modelled through omission, inexperience, etc. Practical guidelines should outline conditions, assessments, warnings, etc. and not prescriptive but guidance in approach. Frameworks provide additional guidance and may include not only direct analytic provisions but also require comparison and/or use of some historic parametric and experience "rule of thumb" provisions. Direct analytic approaches require explicit identification. The "unknown unknown" as well as lack of documentation of limitations is of serious future concern.

C by A.Mangiavacchi: In previous standards, there were minimum requirements that should be fulfilled, e.g. minimum thickness not less than 1 inch, those requirements could prevent failures. This robustness may be lost with modern optimized design.

Discussion related to P-S5-2:

C by S. Moxnes (SM) :Fortunately, the structural design of current platforms within Statoils portfolio seems to satisfy this requirement with regards to global capacity. It is expected that this will be the case for most NCS installations. The largest collision risk is believed to be related to penetration of well conductors or gas risers. .(Ed. comment: See also documentation in the paper P-S5-2)

C by G. Ersdal (GE): Operators are also recommended to try to implement solutions to reduce the displacements of operating vessels and possibly also introduce speed limits for attendant vessels, in particular, when it is very expensive to achieve a structural design which satisfies the requirement

Q from NN: Moxnes mentioned that the number of engineering hours must be reduced. Based on the new requirement of 50 MJ collision energy, how could this is done?
A by G. Ersdal: The existing simplified guidelines for assessing collision damage and fulfilment of ALS criteria, are being further developed, e.g. in a DNVGL project..

C by A. Morandi: Should not blame the junior engineers; there is also a lot of personnel involved, and skilled companies.
A by Abraham: Does not blame the young people, but sometimes wonder how things can happen.

Discussion relating to P-S5-3:
C/Q by Moxnes to M. Singh: Reducing engineering effort could for example be achieved by type approval of equipment for different areas based on dimensioning explosion loads Today advanced tailor made FE-analysis is as an example carried out for a fire cabinet to check for capacity against explosion.
A by M.Singh: Could standardize, but all platforms are not similar, pressure and duration of pressure pulse, and natural periods of platforms.

Editorial comment (by T. Moan): This is true, but experiences in connection with risk analyses to determine for instance explosion pressure time histories and fire scenarios for platforms on the NCS show that there is a potential to use more “prescriptive” requirements for such accidental actions, but that the action effects need to be determined case by case.

Session 6: Reliability of Concrete Platforms

Co-chairs: J. Moksnes; C. O'Brien

P-S6-1: Reliability of a Concrete Floating Barge – the NKP Case (see also associated article: A-S6-1)
P.l Collet, Total, France

P-S6-2: Experiences with the Safety and Durability of Concrete Offshore Platforms
K. Høyland and H. A. Rogne,, Olav Olsen AS, Norway . Presented by H. A. Rogne

P-S6-3: ALARP in Decommissioning Brent D (Cancelled)
F. Lange, Shell Global Solutions

Discussions:

C from Y.S. Choo: There are two types of aging, material aging and aging of personnel. We should avoid making the same mistakes as the predecessors and therefore we should cautiously ensure that engineering knowledge about concrete is conveyed to the next generation. (Degrading of the knowledge of concrete due to limited activities in Norway since 1995 ?)
A by H.A.Rogne: The strength of concrete increases with age. General concrete competence is maintained since there are many concrete projects. However, specific competence e.g. related to loading on concrete platforms could be lost.
A by P. Collet (in writing): in last ten years, many concrete structures have been launched, from the Monaco Floating Dike to the Hebron platform this year, as well as Sakhalin, etc. Knowledge is worldwide spread and share and in Norway. Engineering is still efficient if you consider who were involved in these last concrete projects.

C from S. Haver: I wonder about the removability of the concrete designs of offshore fixed platforms in Norway. The oldest platforms cannot be removed. What about the newer platforms? It was a pity that design life of Troll is 70 years.
A by H.A. Rogne: The earliest designs were not designed to account for the possibility of removal. In order to remove the concrete tanks, the concrete tanks need to be designed to resist the pull loads. The later designs account for the removability but the operators do not intend to remove these platforms yet.
Comment by P. Collet (on writing): I don't know about the Troll platform in particular, but structures designed in accordance with e.g. NS 3473 may have longer life than the planned design life. As concrete is a dead material, removing rock from the sea should be not the only choice. Leaving the concrete structure could be an option with benefit for the environment or fishing. Precaution should, of course, be taken regarding navigation.

Q from G. Kuiper: In view of the development of technology, by now, did concrete concepts become economically competitive?.
A by chairman J. Moksnes: The competitiveness needs to be discussed case by case.
A by H.A.Rogne: The application of concrete can be economically competitive for concepts which makes use of the special advantages of concrete platforms such as the big storage capacity compared with the steel structures.
A by P. Collet (in writing): Competition should to be seen in the light of the number of contractors worldwide, that can built such platform, in steel or in concrete. Engineering and management require a high level of competence but 80% of concrete construction work does not need a high competence

level. No yard with lifting tools is required. This fact changes the way to think about the project and that may be the biggest step for our industry.

Q from T.Moan: It is interesting to note that it can be demonstrated the payload capacity can be significantly increased for existing platforms. What if the modern analyses methods had been used in connection with the original design? Would you then be able to increase the payload capacity if needed later in the service life? Moreover, how do you look upon possible optimization of the initial design versus having som robustness with respect to future changes?
A by H.A.Rogne: The structures are more optimized now. But more load effects are also included, e.g. ringing. Capacity was shown to be ok by using more advanced (non-linear) analyses. The ALS/ULS compliance of Draugen in significantly increased wave/ringing loads, which are not realized at the initial stage, is a good example.

Q from G. Kuiper: Could concrete platforms be made cheaper in order to compete with the alternatives?
A by H.A.Rogne: A steel platform cannot be replaced by a concrete platform, it will be more expensive. However, a concrete platform could replace several other platforms, and in particular if oil storage is needed, concrete platforms could be competitive. (Needs to be included in early design.)
A by P. Collet (in writing): Concrete platforms might not be competitive with steel platforms from a CAPEX point of view, but in consideration of CAPEX + OPEX during 40 years, concrete is back in the race. Moreover, the industry should have a feedback on projects in severe environments, like the North Sea, where disconnected option is required and make comparison with GBS solution.

Q from the audience: Why is it that the latest concrete platform was delivered to the North Sea in 1995? Is concrete not competitive anymore?
A by H.A. Rogne: Concrete cannot compete with steel in the sense that it can directly replace a steel structure on a given location. Concrete can be competitive as an alternative concept where you can take advantage of the main merits of inshore completion, float-out and instant installation, large load bearing capacity and oil storage in the caisson.

Final statement by the chairman J. Moksnes (in writing):
I wish to thank the two speakers for their excellent contributions to this Conference. Not only did they manage to keep to the very strict timetable, but they also delivered impressive case records of marine concrete structures with more than 20 years of service. The oil and gas industry has a very heavy bias towards steel. Of well over 1000 marine oil and gas production facilities in the world, only some 50 are made of concrete. The two papers, from France and Norway, demonstrated that marine concrete structures, whether they are floating or gravity based, can be designed and built to very strict specifications. We have heard that regular in-service inspections after decades of exposure have confirmed very good durability, with little maintenance required. Design reviews have demonstrated that the gravity base structures can accommodate significant modifications to the production equipment and increases in the weight of the top-sides. In addition to the long-serving structures dealt with by the authors, large marine concrete oil and gas platforms have in more recent years been built in Australia, Russia, Spain and Newfoundland.

Session 7: Reassessment of Jacket Platforms in Operation

Co-chairs: F. Nadim; J-L. Colliat-Dangus

P-S7-1: Risk-based Structural Integrity Management for Jacket Structures (see also associated article: A-S7-1)
F. Guédé, Bureau Véritas, France
P-S7-2: Reassessment of Offshore Structures: the Geotechnical Issues
Thomas Langford, et al., NGI, Norway. Presented by T. Langford

Discussions:

Discussion relating to P-S7-1:
Q from F. Nadim to F. Guédé: There is skepticism towards risk matrices. Many boxes are irrelevant. Could often jump over boxes, e.g. from class 1 to 3.
A by F. Guédé: I agree that risk matrices are not an accurate measure to assess the risk level. However, they are convenient to set out the risk assessment results to the stakeholders and allow a structural integrity risk analyst to speak the same language as other risk analysts (e.g. for process equipment, machineries,…).

As you know the issue of the development of an inspection strategy is different than the issue of structural assessment. In structural assessment the more accurate is the assessment the better it is. Of course accurate assessment is beneficial for inspection strategy development too, but the most important is to understand how the structural items compare to each other so as to know where to put the inspection effort. Besides, not only the structural failure is of interest, many other drivers e.g. the effectiveness of the corrosion protection system, should be accounted for in assessing the risk level and the resulting inspection strategy. Therefore, in my opinion the risk assessment does not need to be so accurate for inspection strategy, relative risk ranking as provided by risk matrices may be enough to develop a rational and optimal inspection strategy.

Q from A. van der Stap : How is performance of a platform accounted for in this method to develop an inspection strategy?
A by F. Guédé : I do agree that the historical performance of a platform provides relevant information to assess a platform risk level which allows an inspection strategy to be updated. The current method does not explicitly include a scoring rule for platform's historical performance. However, if such a rule has to be included, it would require experience and knowledgeable persons to formalize how comparison of historical and expected performance should upgrade or downgrade the likelihood of failure.
Ed. comment: See also the discussion relating to Session 8.

Discussion relating to P-S7-2:
Q from H. Singh to T. Langford (in writing after conference):: In reference to presentation by C.Swan & J. Tychsen where they are demonstrating a potential higher risk to our assets against 10,000 year cases. How are we positioned from the pile capacity point of view in light of new findings related to ageing, etc.?

A by T. Langford: I think that reliability-based approaches can be used to calibrate relevant safety factors for such cases using appropriate design methods, and effects such as ageing will help to capture the most realistic foundation performance.

Q from NN: Based on your slides where you had shown quite some new insights and work on a number of suction & piled anchors for different asset owners, (we have seen similar assessments for our assets as well). How are we capturing these learnings and insights in our codes & standards?
A by T. Langford: I think that the codes are working well to update with new insights and methods for such issues, but there is of course more work to be done. For instance, it may be relevant to consider different factors for different anchoring types, such as piled anchors, suction anchors and drag anchors.

Session 8: Inspection Planning with Respect to Crack Control

Co-chairs: H.O. Madsen; P. Frieze

P-S8-1: Lessons Learned from Predicted Versus Observed Fatigue of Offshore Steel Structures (Jackets, Semis) in the North Sea (see also associated article: A-S8-1)
O. T. Vaårdal, Axess AS and T. Moan, NTNU, Norway. Presented by O.T. Vaardal

P-S8-2: Guidelines for Probabilistic Inspection Planning of Offshore Steel Structures
G. Sigurdsson and A. Fjeldstad, DNVGL, Norway. Presented by A. Fjeldstad.

P-S8-3: Fatigue Analysis, Lifetime Extension and Inspection Plans (see also associated article: A-S8-3)
M. Birades, Total and L. Verney, Bureau Véritas, France.

Discussions:

Q by P. Smedley to M.Birades:
Are they doing a full probability analysis or a more pragmatic approach? Is the risk ranking performed with or without safety factors (SCF, uncertainty in SN-curves, uncertainty in plate thicknesses, etc.).
A by M. Birades: in the Netherlands, there is a requirement to inspect every five years, hence optimizing the inspection intervals is not possible. Instead, a risk ranking is performed, in order to decide which details to inspect.

C by P. Smedley to all presenters (in writing):
The question I was trying to ask M. Birades but also the representatives from BV, DNV and NTNU in the same session was related to the balance between effort needed and value gained in the apparently different RBI models presented.
The background for my question is that engineers are likely to only have ready access to design fatigue lives for each component/connection. This can quickly give one measure of probability of 'failure' (exceeding design life) and the relative ranking of such items. Stripping out the explicit and implicit safety factors on load combinations, SCFs, SN curves, etc. will give a different measure of probability of 'failure' (exceeding mean life) and a different relative ranking of such items. A third proposal (DNV) seems to recommend performing simplified structural reliability assessment on each component capturing the uncertainty in each variable, which will give a third measure of probability of 'failure' (structural reliability) and yet another potentially very different relative ranking of such items in terms of risk.

Can the presenters comment on the different approaches and whether one standardised fit for purpose RBI approach should be proposed?

A by O. T. Vaardal and T. Moan (in writing)
We might envisage various measures of risk; i.e. relating to cracks that need repairs - implying operational costs; or the risk of fatalities (and environmental and economical consquences) that would relate to global loss of structural integrity.

For this reason your question is answered in two steps - firstly we deal with the likelihood of crack occurrence and secondly the risk relating to total loss.
Ranking of the component fatigue failure potential

- The probability of fatigue failure based on design fatigue calculations will most likely not correlate perfectly with the observed fatigue crack growth. This is due to uncertainties in the analysis method and deviations in the as-fabricated versus as designed structure. The high degree of variation in local geometry and fabrication quality can be of the same importance as the differences implied by using different fatigue design factors (ranging from 1 to 10). Moreover, there are differences in the procedures used in the global fatigue analysis (load effects). Some of these uncertainties are generic and some are dependent on the particular structure and personnel involved. Only the generic uncertainties can be accounted for in the design and initial planning of inspection.
- In fatigue life estimates during design the uncertainty in fatigue strength (in terms of SN curves) is accounted for by using characteristic values. We might say that the fatigue load effects (stress ranges) are calculated by expected values – with no account of the uncertainties in gloabal load effects, stress concentration factors etc. By accounting for all the systematic and random uncertainties in a probabilistic (RBI) approach, the fatigue estimates will be improved, and the more the specific character of the type of structure, loading, local geometry determining hot spot stresses, are accounted for. Still, a ranking based on an RBI approach is of limited accuracy..(Ref. DNVGL RP C.210 for recommendation of COV value as function of level of detail and quality in the FLS analyses).
- As mentioned, fatigue crack growth is highly affected by local geometry. Hence, we need to include information from an As-Is inspection to get realistic estimate of the fatigue crack growth potential. Based on the As-Is information we select mean value and COV for the SCF and fatigue capacity given by the SN-curve. The DNVGL RP C.210 does not focus the importance of the As-Is inspection. At the design stage we need to use generic information also regarding the effect of inspection – implying predicted outcomes of the inspection.

- But a more realistic ranking requires use of the information from the inspection at the fabrication stage and in-service information. Experience also indicates that realistic ranking of the fatigue failure probability should be given after collection of information from the first in-service inspection.

- In principle, strengthening of the structure or modification of the inspection program might then be relevant. Experiences show that the consequence might be a reduced or increased inspection program.

- The presentation and related paper focus on estimating the probability of fatigue failure (normally defined as a visible crack or similar) i.e. the component failure potential. A true risk ranking requires a combination of failure consequence potential and component failure potential, as partly reflected in the conventional fatigue requirements in terms of allowable fatigue damage or fatigue design factor, in e.g. Norsok, dependent on the consequences of failure and the inspection approach. The failure consequences are differentiated depending on whether ALS criteria are fulfilled after fatigue failure or not. A similar differentiation of targets is also made in a probabilistic approach, e.g. Actually, there are at least three levels that might be considered in this connection for framed structures; namely “fatigue failure”(in the SN- sense referring to visible cracks; through thickness crack (that can be detected by LBB); member failure (rupture of the member). In the ALS approach (for jackets) mentioned

above global failure approximately related to a global ultimate failure and fatigue failure of a tubular joint

- In our presentation and related paper, we have limited our discussion to the failure consequence potential related to the operation regularity. Fatigue failure is detected by the leak detection system. It is a high damage robustness present. Additional crack growth from 2 winter month is required before the global load bearing capacity is affected. Hence, it is sufficient time for planning and implementing the required repair.)

A by Michel BIRADES (in writing): To complement my initial answer to Phil Smedley's question: "The effort needed for implementing a full probabilistic approach seems disproportionate given the uncertainties to be managed and regarding the value gained, it is little considering the statutory requirement (inspections every five years).
This is why TOTAL and BV have presented a pragmatic approach for inspection planning and have underlined the need to properly address the data and computation management (refer to the marine growth thickness assessment and the implicit linearization in the fatigue analyses)."

C and Q by O T Vaardal to M. Birades and L. Verney (in writing)
Unfortunately we did not manage to get time to discuss your paper "Fatigue Analysis, Lifetime Extension and Inspection Plans" during the OSRC2016.. I do really appreciate the focus on the effect of marine growth.

Here I would like to address the inspection scheduling methodology and ask for some clarification of the term "Node Fatigue Life", "Past Fatigue Damage", "Future Damage" and "Lifetime" used in Figure 1 and Table 10 as well as "service life" used in Section 6.4 and "fatigue life" used in Eq. 14.

My interpretations for clarification and questions are as follows:
Clarification 1 (Q by Vaardal):
"Node Fatigue Life" = "Fatigue Life" = "Lifetime" and is year in service before the accumulated fatigue damage reach 1.0. Hence, the scale of the horizontal axe in Figure 1 is 0.0 at time of installation. The value of "Node Fatigue Life" = "Fatigue Life" = "Lifetime" will vary as the past damages are adjusted by measured thickness of marine growth.
C by M. Birades: Yes, the node fatigue life is the year in service before the accumulated fatigue damage reach 1.0 and the scale of the horizontal axe in Figure 1 is 0.0 at time of installation. The node fatigue life depends on the measured thickness of marine growth in the past, but also on the extrapolated thickness of marine growth in the future if the node fatigue life is above the present date.

Questions 1 and 2:
Is the calculated Node "Node Fatigue Life" = "Fatigue Life" = "Lifetime" including any Fatigue damage design factor as given in Table 6? (I assume they are not included in the value of "Node Fatigue Life" = "Fatigue Life" = "Lifetime" as these factors are used in the inspection scheduling methodology described in Section 6.4) If they are used, how is the factors in Table 6 related to "Not Inspectable", used?
A by M. Birades: For inspection scheduling, the node fatigue life shall not include any Fatigue damage design factor as given in table 6. Nevertheless those factors can be used if the objective of the fatigue analysis is to check if the platform copes with the requirements of the standards.

Clarification 2 (Q by Vaardal):
From Section 6.4

1) If ranking score is smaller or equal to 5 times the service life the weld is subject to inspection. The safety factor of 5 covers inspection of secondary welds in case fatigue life is equal or smaller than 2 times service life.
My interpretation is that the requirement of 5 times the service life in ratio is from Table 6 for the combination Inspectable & Failure critical component. The use of factor A=2.5 for secondary weld is related to factor 2 in Table 6 for the combination Inspectable & Not failure critical component i.e. 5/2.5 = 2. The use of A=10 for tertiary structural elements is related to use a Fatigue damage design factor of 0.5 (=5/10).
A by M. Birades: There is no link between those different numbers. The numbers 1, 2.5 and 10 of Table 7 have been elaborated before API and ISO provide Table 6. It is only a coincidence…

Clarification 3 (Q by Vaardal):

Ref- Section 6.4

The minimum inspection scope is:

- 5 year interval between the inspection campaigns
- at least 5% of the primary welds to be inspected every fifth year

As long as this minimum requirement is fulfilled, the scope for an inspection campaign is a random selection of 20% of the hot spot given a ranking factor less than 5 times the service time. During the service life of the structure, all primary welds given fatigue life longer than 200 years and secondary welds given fatigue life longer than 80 years, will not be considered for inspection during the entire service life of 40 years.
A by M. Birades: Correct!

Question 3:

If we during an inspection campaign detect a large number of fatigue cracks e.g. 20% of the planned inspections results in fatigue crack detection, will the applied methodology recommend an extension of the scope of inspection? Is it defined any limit of crack detection during a campaign before changes in the inspection program is required? (Factor or weighting index C and D are related to the actual hot spot or weld. Is there any overall weight index for the entire structure?)
A by M. Birades: There is no defined limit ; this shall be considered case by case. I must say that, fortunately, such a scenario has never occurred.

Session 9: Reliability of Mobile Units

Co-chairs: O. Dalane, C. Wang

P-S9-1: Operational Experiences and Design Codes for MODU
T. Sildnes, DNVGL, Norway

P-S9-2: Reliability of Jack-up Platforms
M. Hoyle, DNVGL

Discussions:

Q from O. Dalane to M. Hoyle: How are the class rules for jack-up platforms related to ISO-requirements?
A by M. Hoyle: They are not too far apart, but not in the same place either.

Q from G. Kuiper: Has there been a change in air gap requirement after the COSL Innovator accident?
A by T. Sildnes: There are no changes in the rule requirements. Negative air gap is allowed if the deck box is designed for the horizontal loading. However, following the accident, DNV GL has developed two new guidelines for column-stabilized units for prediction of air-gap and horizontal slamming loads. (Offshore Technical Guidelines OTG-13 and 14). DNV GL classed units operating in harsh environment are required to re-assess air gap for ULS condition according to OTG-13. If a negative air gap is found, it has to be documented that the deck box has structural capacity to withstand horizontal slamming loads. Norwegian Petroleum Safety Authority PSA and Maritime Authority (NMA) have also issued letters to the industry stating that column-stabilized units under their jurisdiction have to be re-assessed with respect to air gap and slamming strength according to the new DNV GL OTG documents.

General reflections of the session, by the co-chair (O. Dalane):

Two very good and informative papers were presented in this section. The papers were closely linked and complemented each other in a very sensible way.

The first paper by Mr. Tore Sildnes gave a good introduction to the class regime including background (origin) and how it works. It was pointed out that accidents have always influenced maritime rules and regulations, and examples of accidents and their impact on the regulations was given. It is acknowledged that the COSLInnovator accident (wave impact in the deck box) also was openly discussed in the presentation.

The second paper by Mr. Mike Hoyle focused on Jack ups and their reliability. It was interesting to observe how site specific assessments and requirements had evolved over years and how the work from SNAME had developed into the ISO19905-1 standard. It was noted that there still is lessons to learn that need to be implemented into the ISO standards and that most of (or all?) of the jack-ups are classed according to a class society and have to follow their codes.

Session 10: Stability of Floating Platforms in a Reliability Perspective

Co-chairs: J. Stear; T. Moan

P-S10-1: Assessment of Intact and Damage Stability Regulations for Offshore Floating Structures – in a Reliability and Risk Perspective
C. Wang, ABS

P-S10-2: Reliability of Floating Platforms with respect to Stability
D. E. Engberg, DNV GL, Norway

Discussions:

Comment by the chair (T.Moan):

The two interesting presentations made in this session complement each other in that the first one focuses on analysis and design while the second one focuses on operational issues relating to stability issues such as ballasting, weight control and watertight integrity.

In the first presentation it was emphasized that most of the "stability/sinking" failures that have occurred in the offshore oil and gas industry do not seem to be due to inadequacy of the existing criteria, but due to other risks that are not part of the current stability calculation methodology. In almost all cases multiple failures were necessary for the results to be catastrophic

Stability criteria for offshore structures are traditionally based on a deterministic, quasi-static approach. Dynamic-response-based approach has been developed but is adopted at difference pace by class societies, IMO, and local regulatory bodies. While reliability methods are not applied, risk-based approach is gaining its way to the damage stability assessment. However, more work on a better understanding of the actual failure causes/mechanisms and the related consequences, and how to estimate them in risk assessments, is needed.

In the second presentation human factors associated with the operations were emphasized, suggesting increasing the competence and awareness through damage control drills.

It is also important to establish a tight connection between assumptions made in design regarding keeping watertight boundaries closed at sea and operational procedure manifested in the operational manual.

Session 11: Reliability of Floating Platforms

Co-chairs: A. van der Stap; T. Vestbøstad

P-S11-1: Industry Standards for Integrity Management of Floating Systems
J. Stear, Chevron, USA. Presented by A. Mangiavacchi

P-S11-2: ALS Design in Practice – Floating Platform Application
R. Løken, Aker Solutions, Norway

Discussions:

Discussion related to P-S11-2:

Q from H. Singh to R. Løken: Analyzing ship collision assuming 'worst case' scenarios may be overly conservative, especially on floaters where as much as 60 % of the energy may go into motions of the floater.
A by R. Løken: For spars, there is a large effect of tilt. For semi submersibles, there will be induced yaw motions, absorbing collision energy.
Ed. Comment: See alsos A-(P-)-S5-2.

Q from S. Uto: How important are viscous effects on the wave drift forces on ships compared to Semis?
C by Moxnes: May be even more important for ships. Wamit results will underestimate the wave drift forces especially for flared bow shapes in high sea states.

Question by C
Q from R. McKenna: The ALS case for floaters, is it better or worse than for fixed? Is the reliability level the same?
A by R. Løken : Wave in deck is worse for fixed than for floating structures. But progressive failure, e.g. capsizing, is not checked. Only the ALS case is checked.

Additional Comment by T. Vestbøstad: What is the relative importance of the ALS requirement to floaters vs. fixed platforms is difficult to say, since it will be case by case dependent. For instance, wave in deck is more "absolute" for fixed than floating platforms.

Final remarks by co-chair A. van der Stap:
Thank you for your good presentations: Jim Stears focusing on managing the risks of life extensions of ageing floaters and Rolf Løken on the design conditions of the same.

After Rolf Løken's explanation on the risk for water on the lower water deck on equipment and people one unanswered question which stuck with me was: "why would we as an industry want to accept a negative air gap for floaters?". Shouldn't the starting point be – no negative air gap and only after ALARP risk assessment accept a deviation by exception.

Session 12: Reliability of Ship type Production Units

Co-chairs: T. Sildnes; C. Wang

P-S12-1: FPSOs in Harsh Environment. Status and Main Learnings after 30 years' Experience with One of Our Most Robust, Reliable and Flexible Concepts
E. Hovland, J. I. Dalane and H. Mong, Statoil, Norway. Presented by E. Hovland.

P-S12-2: Design Basis of World's First FLNG to Achieve Good Reliability
A. van der Stap, Shell Global Solutions, the Netherlands

Discussions:

Discussion relating to P-S12-2:

A by Van der Stap in response to a question from the audience: the design criteria are such that damage is accepted for 10000 year waves. For 40 year wave there should be no mechanical damage. For 20 year wave, the production should continue.

Q by S.Haver (SH): How is the "Response based design" implemented?
A by Van der Stap: Using in-house models, creating long term distribution of mooring line loads.
SH: Seems to be similar to what we call "Long term response analysis"

C by P. Tromans: The comment was in response to a discussion relating to damping of horizontal motions of the FLNG in the long-term response analysis.
Comments on viscous effect and model test : we account for viscous damping of roll and horizontal motions of the floater from viscous drag forces on the hull and mooring lines through empirical models. Where possible, we have updated the models based on the various model tests that have been performed.

Q by Y.S. Choo: in side by side operations and with respect to the offloading arm, what is the limiting sea state?
A by Van der Stap: there are about eight criteria, like roll and pitch motion, clashes, mooring line loading etc. The limit also depends on the wave period, but it is around 3 m Hs.

Q by C. Wang: Is there a limitation with respect to offloading ships (for the FLNG)?
A by Van der Stap: No, all kinds of offloading ships can be used.

Comments from co-chair T. Sildnes:
In the session, there were two very good presentations about FPSO concept solutions for offshore field developments. They were particularly interesting for me representing a classification society, since FPSOs combine regulations, technology and practices from both the maritime and offshore platform world.

The first presentation by E. Hovland from Statoil provided a very interesting experience feed-back from operation of harsh environment FPSOs over two decades. It is reassuring to learn that FPSOs combine fast track development with robust and reliable operation, suitable for life extensions up to 40+ years without leaving location. We also see that FPSO concepts are the preferred solutions for

future field development projects in harsh environments in Barents Sea and East Canada. The improvement areas presented provide important input for updating of FPSO rules/standards.

The second presentation by Andre van der Stap gave an interesting overview how design basis has been established for the world's first FLNG facility. It was particularly interesting to learn that the unit is designed to cope with extreme cyclone weather conditions on location without disconnection.

Session 13: Reliability of Station-keeping Systems

Co-chairs: P. Smedley; E. Hovland

P-S13-1: NorMoor JIP – Mooring Design Code Calibration
S. Okkenhaug, DNVGL, Norway

P-S13-2: Reliability of DP Systems (see also associated article: A-S13-2)
H. Chen, Lloyds Register, China

Discussions:

Q from P. Tromans to S. Okkenhaug: Though it is very accurate for more than 90% of cases, it seems from some model tests and other observations that, for a few cases, conventional linear and second order diffraction analysis occasionally does not work very well in predicting wave induced floater motions and consequent line tensions. Should this appear as an additional safety factor in the design code or be treated in another way?
A by S. Okkenhaug: It should be treated separately

Q from S.Lacsse to S.Okkenhaug: Do you refer to annual or service life failure probabilities?
A by S. Okkenhaug: Annual values

Q from A. Duggal to S.Okkenhaug: In your LRFD approach do you apply the load factor on static action on the pretension or to the mean load?
A by S.Okkenhaug

Reflections of co-chair (P. Smedley):
In this session we received two new presentations on station-keeping systems, one on moorings and one on dynamic positioning. Ms Okkenhaug provided a brief summary of the extensive work ongoing and planned in the NorMoor JIP covering notional mooring reliability in a wide range of global environments for both drilling and production floaters. That the proposed headline factors of safety are unlikely to change significantly from those used by the industry for decades is reassuring and, along with the other findings, provides significant risk assurance for our industry mooring systems which need to be supported through construction, installation and full life operation. Dr Chen's presentation on DP systems is my first sighting of quantitative data on DP system reliability, while it is reassuring that DP collision risk in direct offloading concept appears 2 orders of magnitude better than tandem offloading systems, we should not become complacent. The consequence of DP failure in benign conditions is likely to be small and thus likely underreported compared to tandem offloading. His recommendation for independent reporting and collation of data would be most helpful to operators if it can be achieved.

Session 14: Floating Arctic Structures

Co-chairs: P. Liferov; M. Maes

P-S14-1: Sea Ice Management and Reliability of Floating Structures
R. McKenna, McKenna & Associates, and B. Wright, B. Wright Assoc.,Canada. Presented by R. McKenna.

P-S14-2: Applying the Limit State Definition in Ice Class Rules for Ships to Offshore Structures (see also associated article: A-S14-2)
K. Riska and R. Bridges, Total, France. Presented by K. Riska

Discussions:

Q from co-chair P. Liferov to both speakers :
From the presentation of Richard McKenna it follows that the safety of arctic floating structures depends on many barriers (design and operational measures). At the same, Kaj Riska presented significant "variabilities" in the definitions and design applications. Kaj Riska also emphasized that "one should better know an answer beforehand", pointing to the fact that experience is vital. With this in mind, what is your message to those involved: academia, designers, operators and not least regulators? Is there a challenge since a lot of the (ice) experienced engineers are retiring and there is a significant (20+ years) gap?

A by K. Riska:
The standards should be more explicit. ISO 19906 is in parts unclear and difficult to implement. This is especially the case for less experienced engineers. Thus there is room for improvement. There are differences in the approach of ship and offshore structure rules for ice actions; these rules are not commensurable and this makes the use of ship and offshore structure rules challenging for floating structures. This difference exists in both local and global actions. Care should be exercised when extrapolating from measured / observed data for return periods of 100 or 10 000 years. The ad hoc nature of the methods given in the ISO 19906 Arctic Offshore Standards in its informative part causes confusion and also a large scatter in suggested design values.

A by R. McKenna:
- Comments on ice pressures: ice pressures in ISO 19906 for the calculation of actions on offshore structures are included in an informative annex. This is an acknowledgement that a single approach cannot apply to all types of structures, ice conditions and loading situations. While a number of relationships for ice pressures are given in ISO 19906, the main message is that "appropriate data shall be used for the specification of ice actions", whether or not these are given in the document. Although ISO 19906 deals with actions from icebergs and sea ice, the present discussion involves only sea ice. Can the ice pressure specification in ISO 19906 be described as ad hoc as compared to the one in the Baltic rules? The basis of each is different and their purpose is also different. In effect, both could be considered ad hoc. Ship rules should reflect the different ice conditions a vessel is exposed to within certain geographical regions. A transiting ship tends to avoid the worst ice conditions, while an icebreaker sometimes has to deal with the worst ice conditions. A structure needs to deal with all ice that passes by. The user of the ISO 19906 standard is free to apply ship rules for estimating ice pressures in floating structure design as long

as it can be demonstrated that they are appropriate and that the expected range of ice situations and features interacting with the floater have been properly addressed.

- Operational procedures and their effect on ice pressures: floating structures are typically supported by ice management (icebreaking) vessels and are often disconnectable. The effect of these factors on the actual ice conditions and features reaching the floating structure can potentially have an effect on the hull design strategy. Although operating criteria for floaters in ISO 19906 (particularly in the ongoing revision) can limit the exposure of the structure to the most severe ice conditions (in terms of thickness and floe size), there is a finite probability that the operating criteria are not met. In such cases, the ice pressures on the hull can be limited by the capacity of the station-keeping system, by what is termed pack ice pressure (when the ice cover is in a compressive state), and potentially when the floating structure is in a disconnected state (whether intended or unintended).

Q from S. Uto: There is a large difference between standards. The standards should be based on experience. Need a feedback loop. How could we try to harmonize the offshore and ship rules?
A by K. Riska: The harmonization starts from having an adequate theoretical basis for assessing ice actions. As most of the methods for ice actions are empirical and of an *ad hoc* nature, there is still a long way for scientists to reach an understanding of ice actions; a sound theoretical basis is a good target for research and also for collaboration between those who develop knowledge and those who apply knowledge.

Q from A.Morandi: The load factors for ice loading are similar to factors used for wave loading. How come?
A by M. Maes: The factors are based on a compromise between several items, in order to be generally applicable. For smaller geographical areas and specific structural systems, the factors could have been different.

Comment by R. McKenna: Pressures averaged over larger areas for global actions will have less variability than local pressures used for specific parts of the hull.

Q from Y. S. to K. Riska: What is your advice to universities as it is difficult to have simultaneously practical experience and theoretical expertise?
A by K. Riska: This is a good and also a common question. There is no really a good answer. However, universities could cooperate with industry and include practical parts in a more theoretical curriculum. These parts could be given by people having practical experience. Secondly universities should give emphasis and support to acquiring theoretical methods and knowledge to engineers as industry will not always be willing to do it for them. A balance in the curriculum is naturally difficult to obtain as there is a lot of pressure nowadays for including a lot of general studies.

Session 15 - on ISO 19900 WG1 ongoing work

Co-chairs: M. Maes and H.O. Madsen

Background: Developing a third edition of ISO 19900 by WG1 (M. Maes, M. Abraham, S. Balasubramanian, M. Birades, P. Frieze, M. Hoyle, K. Høyland, A. Mangiavacchi , T. Moan, S. Moxnes, P. Smedley, J. Waegter, D. Wisch)

The session consists of presentations and discussion of the preliminary results of the work of technical panels on:
P-S15-1: Limit States & System Effects (see also associated article: A-S15-1)
T. Moan

P-S15-2: Risk, Consequence, and Reliability Classes/Targets
P. Smedley

P-S15-3: Uncertainty Assessment (see also associated article: A-S15-3)
P. Frieze-lead

P-S15-4: Lifetime Extension
S. Moxnes

Discussions:

H. Madsen: There seems to be a lot of uncertainty related to ISO 19900 and ISO 2394, based on the presentations given.
T. Moan: The "motherhood" ISO 2394 standard is a reference for ISO 19900, but it is made by "civil engineers" for onshore structures; and hence does not cover stability and mooring of floating structures.Improvements on such aspect are also needed in ISO19900. But, the situation with the ISO standard 19900 is after all not that bad.
P. Smedley: Unlike ISO 2394 and other 19900-series standards, ISO 19900 can be read front to back to understand the basic design and assessment principles, although the main audience are the authors of our other ISO structural standards. It is currently not perfect, and the Limit State section, in particular, can be improved in terms of clarity. ISO 19900 cannot contain everything, but needs to refer to other platform-specific standards in ISO 19900 series. It should be remembered that this ISO standard will define the principles to be used the next 20 years or so, and feedback from users is essential.

H. Madsen: With a large amount of work, what is the ranking of priorities in the committee?
M. Maes: First draft to be sent to the group early next year. The standard should be published by end of 2018.All problems will not be solved with the next revision, but the standard will be improved. Maes does not like the term Alphabet-soup that is sometimes used with reference to the groups and categories defined in the standards. When we are designing such large and expensive structures, we should not mind the small effort involved in finding the most accurate risk/consequence categories applicable to the structure.
A. Mangiavacchi: Agree, it should not be too simple, but as simple as possible. Important to have clear definitions.
S. Moxnes: Some things could perhaps be made simpler. For example, with regard to exposure levels, the categories are most often L1 or L3, and more seldom L2.

H. Madsen: Will the next ISO 19900 be decreed by law in any countries?
P.Smedley: My understanding is that ISO standards can only gain legal basis if specifically referenced by the regulations in that Country. National Annexes in ISO standards can be included where a Country seeks more onerous requirements and can also informatively reference such legal requirements. Where an ISO Standard is adopted by a Country's national standards body (e.g. Standards Norway) then there should not be any other national standard that covers the same scope, i.e. the ISO standard gives the single source of information. ISO 19900, has been widely adopted as evidenced in an IOGP publication which maintains the list of ISO oil and gas standards global adoption, e.g. in Europe, Brazil, Canada, Russia, China Kazakhstan and Gulf States.

Ulf T. Tygesen (Ramboll Oil & Gas): Does the new code allow for and will it be prepared for utilization of advanced Structural Health Monitoring Systems (SHMS) methods for Bayesian based wave load calibration (probabilistic FEM updating) to field measurements and prepared for quantification of uncertainties in terms of estimation of Coefficients of Variation (CoV) and Bias values for linking performance of structural prediction models against SHMS measurements with Risk- and Reliability based Inspection Planning (RBI) and final verification of the "actuarial" safety level (PoF).
P. Frieze: Monitoring generally requires advanced equipment.
S. Moxnes: Chapter 12 in ISO 19900 allows the use of measured results in the assessment of existing structures.

J. Tychsen: In Denmark the law requires that all risk related to life safety and severe pollution are "Identified", "Evaluated", "Reduced" (ALARP), "Documented" against "Pre-defined" criteria and "Informed" to authorities (HSR evaluations). It is not common industry practice to include risk from extreme environmental loading and fatigue in the HSR. Risk contributions from these sources have been considered negligible as structures are in code compliance. If the structural code is performance based and hence does not carry an official ISO reliability based ("*actuarial*" or "*notional-conservative*" if we cannot make a sufficiently accurate actuarial reliability model) calibration we cannot evaluate our risk. Having recently looked into the code background data our present conclusion is that compliance with the structural code (ISO 19900 series) cannot document formal legal compliance in Denmark (DK) as we do not know the inherent actuarial or "*notional-conservative"* safety level. NB. From what we see, UK and DK legislation seems to have much in common on the safety case issue, i.e. the above legal requirements are likely not unique for DK but could be relevant for several EU countries.
P.Smedley: ISO standards are the consensus opinion of all participating members and are written to be used in all Countries. Therefore, it cannot cover the special needs and requirements of any one Country unless agreed by all other member Countries (see above comment about inclusion of National Annexes for specific additional requirements). In terms of guaranteeing a specific level of reliability in our Standards, that is very challenging as discussed in my Keynote presentation. In benchmarking factors of safety in ISO 19906, many combinations of variables where considered to ensure that a 'not to exceed' reliability limit of 10^{-4} was achieved. Still, despite this extensive study, it is feasible that a combination of variables that were not considered could be found to give a higher notional probability of failure.

P. Smedley: We understand the desire for more background to the safety factors in the 19900-series Standards but need to be careful as, in the past, background reliability values have become publically misquoted by some readers.

P. Frieze: When SRA are run, it seems that all engineers get their own result due to different interpretation of input etc.
T. Moan: The notional target level referenced in structural reliability analysis is different from the actuarial values. Moreover, it is important that we are not using generic target values – they need to depend on the geographical region. Finally, it should be noted that the current action factors are not consistent in view of the uncertainties affecting actions and action effects in various types of offshore structures and intended (structural) reliability target levels.

J. Tychsen: We are concerned about the inherent risk levels in the ISO 19900 series of codes. ISO 19902 specify eg. 10000y ALS return periods (no safety factors) and in the proposed draft for ISO 19901-9 this is proposed reduced even further to 2500y. Taking UK and DK ALARP guidance this is far into the ALARP (”tolerable”) region and for manned operation close to the “Intolerable” border. Here it should be kept in mind that the intolerable border covers all accumulated risk and not only the environmental risk, i.e. fire, explosion, boat impact, fatigue collapse risks to be added to the environmental overload collapse risk. Being far into the ALARP (“Tolerable”) region operation is allowed but conditioned on all reasonable means are applied to reduce risk to ALARP (e.g. actions like strengthening of critical failure modes, load reductions, adverse weather de-manning etc. must be evaluated). Question: Is it not problematic that the code stipulates so low return period ALS criteria without requiring combination with other risks nor set forth any requirement for ALARP initiatives? It is noted that ALARP considerations are already included in Section 24 in ISO 19902, i.e. the principle is not new in the context of the ISO 19900 series of codes.

P. Smedley: Currently, according to ISO 19902, the ALS case requires an air gap check at a return period between 10^{-3} and 10^{-4}. Maersk has suggested that by following the ISO 19902 standard can lead to notional or actual probability of structural collapse less than 10^{-4} p.a. This would be a concern to many of our participants and we would welcome further information that could lead to improvements in this Standard, if that is the case.

Y.S. Choo to Moan and Moxnes: The Bomel benchmark analyses used 3 steps in the tests regarding frame components and ultimate capacity. It is only valid if the designer also uses common sense.
S.Moxnes: The competence is important when using the standards.
T.Moan: I am not sure what you mean. But if you refer to my presentation, I showed a Bomel test result for tubular joints to show that the limit for ultimate strength should be carefully chosen when parameterized formulae for the ultimate strength of tubular joints for design standards are established. The point is that ultimate failure might imply large plastic strains before maximum capacity is reached. For structures under cyclic loading the maximum capacity determined by static analysis might not be reached in the presence of fatigue cracks. As there is no explicit limit state for ultimate fatilure following farigue crack growth, this fact needs to be recognised by code writers (since it is beyond the features that designers can deal with). This is not new insight –as I mentioned, it was pointed out prof. Yura in developing the API strength formulations in 1980. Moreover, such a

consideration also applies when nonlinear FE analysis is used to determine the ultimate strength of components and system (e.g. in pushover analysis of jackets). See also paper and presentation S15-1.

S. Lacasse: The foundations of offshore structures have 'special needs'. It is often referred to structural failures, but avoiding structural failures is not sufficient if there is a foundation failure. How can the ISO accommodate the special needs for foundations?
S. Moxnes: The foundation is considered to be a part of the structural system. At the 2012 Offshore Structures Reliability Conference a presentation made by Puskar concluded that "few if any" of a total of more than 250 platform failures due to USGOM hurricane events were caused by foundation failures.
S. Lacasse: Should keep in mind that there are many other types of foundations than piles. How will ISO account for the geotechnical aspect?
P. Frieze: In ISO 2394, there is a separate annex (Annex D) for geotechnical structures.

R. McKenna: The ISO 19900 standard should bring us forward, not back to the 1980's. What I mean by this is that it should do more than just address common material from the various ISO 1990X series standards. It should seek to explore a workable path forward in terms of consistent limit states categorization across all documents. In its presentation of limit states, ISO 19900 should promote categorizations that adequately reflect life safety and environmental risk, even when they potentially conflict with past practice. In such cases, one needs to acknowledge current practice but provide the necessary guidance to nudge this practice to reflect present thought. On the issues of foundations and robustness, we should be pushing gravity base structures toward sliding rather than overturning.

T. Moan: I think that it is important to explicitly deal with robustness in structural design. Robustness should not be used synonymously with redundancy. A structure can be redundant and still not being robust. For a jacket, several members can be overloaded at the same time under extreme wave actions, and the effect of apparent "redundancy" is lost. On the other hand, a single RC tower with thick concrete walls, might be robust with respect to many hazard scenarios, even if it is not redundant in terms of components. Robustness is crucial in connection with accidental loads and in connection with the effect of inspection and repair to ensure reliability in relation to deterioration phenomena like crack growth. Therefore it is better to refer to robustness in view of relevant hazards and their variation in time and space, rather than redundancy of the structure. See also the presentation (P) and article (A)-S15-1.

R. McKenna: Based on presentations and discussions over the course of the conference, there appears to be a lack of clarity in some definitions (e.g. hazards, extreme, abnormal, accidental) and in the consequences associated with different limit states. Some standards imply collapse, overturning or sinking of a structure to be ULS, while others consider them as ALS cases. It seems to me that we should be pushing our limit states categorizations toward consequences that are similar in terms of severity, whether impairing function or preventing the worst.
P. Frieze: An onshore building is a cantilevered structure, similar to an offshore platform. The robustness must be related to other measures.

Question from NN:

There should be focus on robustness: 10000 year wave. The challenge is which criteria to use. From ISO we find 10000 year. DNV has ALS requirements. 10000 year criterion becomes a design requirement and not an evaluation only. How to help design engineers to prevent progressive collapse.
Frieze: We do not define target 10^{-4}.

A. Duggal: We can design a system strong enough to resist 10000 year loading, but it could still be weak with respect to FLS.
S. Moxnes: There is not a requirement to 10000 return period code check for mooring systems in any design codes.

A. Van der Stap: There is a lot of smart people here. Still, it is easy to lose the overall picture, and focus on small details. We need to think holistic. And is the risk in the Gulf of Mexico sufficiently well understood?
A. Mangiavacchi: What is sufficiently well?
A. Van der Stap: The Alexander Kielland accident is an example of a case that we understand sufficiently well. Generally there is a good track record, but many errors in the GoM are caused by a too small air gap.
M. Abraham: There are several old platforms, even with cracks. They still fulfill the requirements as long as the air gap is sufficiently large. We have probably learned from about 80 % of the failures. Several failures were expected, e.g. caused by a too small deck height.
M. Maes: ISO 19900 will reflect best practice in structural safety. It will also leave room for special needs required by concept-specific standards, e.g. when it comes to limit states for arctic structures and for geotechnical structures.

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