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Introduction

The 2014 *Offshore Structural Reliability* conference was hosted by API for the same purposes as similar past events such as *DIRT (Design–Inspect–Redundancy–Triangle)* conference in 1983, the series of *Civil Engineering in the Oceans* conferences by the American Society of Civil Engineers (ASCE), and the *Reliability of offshore structures* workshop by the Association of Oil & Gas Producers (OGP) in 2012 [now the International Association of Oil & Gas Producers (IOGP)].

Practitioners and end-users of structural reliability methods were brought together for the purpose of sharing the collective knowledge of applying reliability theories and operating experiences in order to address the offshore design and operational challenges facing the industry.

These proceedings contain the material presented at this conference that included alternating sessions of instruction and topical papers starting with the history of offshore reliability studies, progressed to current activities, and then finally outlined issues for future resolution. This event was of interest for operators, engineers, regulators, academics, and anyone else involved in the design and operations of offshore structures.

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Keynote Paper 1

History behind the development of Limit State (LRFD) Offshore Structures Design Standards

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ABSTRACT

The manner in which reliability is applied to structural engineering is different from most other forms of engineering. Other engineering disciplines tend to consider reliability in the context of the performance of many thousands of identical precision made components where physical and performance tests can be used to quantify product reliability. Structures, which may involve many thousands of tonnes of material and site assembly, are not identical and cannot normally be physically tested on completion. Reliability is therefore addressed in the design and fabrication standards.

This paper addresses the early development of structural reliability as a concept, initial applications to standards development and how it has been incorporated in the International Standards Orgainisation (ISO) Offshore Structures Standard.

Offshore Structures designed and fabricated to current standards have in general a satisfactory reliability. An overview of where the industry is in terms of reliability is provided.

INTRODUCTION

The oil industry has built platforms offshore since 1947. Initially they were in very shallow water using dock piling technology. As they extended out into deeper water tubular jacket structures evolved. The oil first industry specific offshore structures design recommended practice API RP 2A was developed in the early 1970's by the American Petroleum Institute (API) focusing on the US Gulf of Mexico. This RP has been actively developed with 20 further editions published up to the mid 1990's. Although written primarily for application to US waters it has been applied extensively to offshore structures design worldwide. With the discovery of oil and gas in the harsh waters of NW Europe other national practices and standards in the UK and Norway were developed reflecting the specific needs of this region.

Initially the objective of the oil industry standards was to address the key design issues such as marine loading, tubular joint design, fatigue, materials selection and fabrication tolerances with a conceptually very simple single WSD load factor. This resulted in an inconsistency in reliability depending on the accuracy with which the extreme marine load can be predicted and the ratio of the live load to the self-weight of the platform. The industry is now in the process of migrating to the slightly less simple but more consistent limit state (also LRFD) method for specifying load factors.

The oil industry has been behind other structural sectors in applying reliability based design. It is fast catching up. This paper outlines some of the history of the development of reliability based structural design standards and the adoption of them by the oil industry.

HISTORY OF STRUCTURAL STANDARDISATION

The development of standards in structural design has been a very long term process. Early examples of standardisation tended to be in the sphere of military engineering. The Romans had a systemised process operated by a corps of military

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engineers for building forts, roads and bridges with variations in standardised design according to the duration of use. An early application of the reliability concept. The Persian King Xerses had sufficient confidence in his military engineering organisation to have ordered it to build a temporary bridge across the Hellespont (Dardanelles) which comprised some 676 identical linked hulls. Chinese military engineers built the Great Wall to a standardised approach to provide long term protection for their civilisation from the Mongols.

Individual ancient civilisations built religious and civic structures in stone in many places around the world. Some such as the Egyptian and Mayan pyramids had a degree of standardisation. Others were unique. The durability of these ancient stone structures is a source of wonder to many archaeologists and historians but is not to an engineer. Stone material that had been chemically stable and subject to high compressive forces for millions of years was quarried and re-shaped to function in a largely compressive mode. The structures subsequently survived with minimal maintenance for a further few thousand years. Failures have primarily arisen from weathering, removal of material for use in later buildings, conflict and in specific regions seismic action rather than classical overload.

Much of the ancient civic and domestic built environment used materials that are not generally highly durable, mud, brick and wood. Failures arose from poor construction, inadequate foundation, fire, earthquake and conflict as well as classical overload. The design attention tended to focus on the decorative rather than structural capability. Over centuries an improved understanding of materials allowed a reduction in conservatism without a reduction in safety.

Perhaps the first time that the capability of materials was deliberately stretched was in the construction of cathedrals in Europe. The great cathedrals were intended to be awe inspiring and were built, often over a period of 100+ years, by masons who observed how the structure performed as it was being built and, if necessary, applied corrections as the building progressed. They deliberately pushed the bounds of materials and knowledge and sometimes exceeded it. Local structural failures were not uncommon during the construction process. Fortunately most were built in zones of very low seismicity. They produced drawings of their structural details which were available as guidance to others building later structures. Bridges were the other area where early engineers were forced to extrapolate knowledge and precedent to achieve longer spans. Masonry arch bridges are inherently tolerant of high loads and foundation settlement. The reliability of arch bridges was first documented in a paper written in 1801 (ref 1). Failures arising from scour undermining the foundations were more prevalent than structural overload. The industrial age introducing iron as a construction material accelerated the process of consideration of factors of safety for appropriate reliability. The application of cast iron in beams and columns in factories and warehouses and wrought iron chain in suspension bridges was accompanied by early scientific papers providing guidance on aspects of reliability. (refs 2, 3, 4)

DEVELOPMENT OF RELIABILITY BASED DESIGN PRINCIPLES

The principles of structural reliability as we understand them today were first evolved in the late 1930's. A M Freundenthal at Columbia University was working in this field for civil engineering structures in 1936. A G Pugsley at Bristol University applied the principles of structural reliability to aircraft structures in 1942. These two published the first papers that outline the concept as it is currently understood. Freundenthal in 1947 published "The safety of structures" (ref 5). Pugsley in 1951 published "Concepts of safety in Structural Engineering" (ref 6). Both of these authors were strongly of the opinion that structural engineer's knowledge of the reliability of structures lags far behind their ability to undertake complex analysis. Both viewed the key need to be improved statistical understanding of the loads to which structures are subject. In a 1956 American Society of Civil Engineers (ASCE) Paper (ref7) Freundenthal provided definitions for unserviceability and failure and outlined procedures for the numerical evaluation of factors of safety. For the next ten years the subject remained mostly in the academic domain in the US and Europe with the meetings of the International Association of Bridge and Structural Engineers being the primary event at which researchers tried to bring together a co-ordinated view.

Historical evidence as a means of quantifying reliability is good at identifying obviously inadequate structures but is not good at identifying directional trends leading to over conservative or insufficiently safe structures. This is because over time subtle changes are made to the processes involved in building a structure. These may be changes in the material production process, changes to the performance and consistency of materials, changes in sampling, quality control and inspection techniques, variances in documentation, changes to site assembly techniques, variations in processes resulting in changes to dimensional accuracy, materials handling techniques that reduce unintended damage etc. These changes, in addition to changes in service requirements, make it difficult to accumulate reliable statistical distributions for the variables used in classical reliability calculations. These changes over time often, but not in every situation, contributed to improved structural performance. In 1966 ASCE published the final report of the ASCE Task Force on Factors of Safety (ref 8). This principally addressed the application of classical probability theory of reliability to structural members and systems. It was considered to be a pioneering report of considerable theoretical interest but to be of limited practical help to any code committee faced with the problem of choosing safety factors. This is because classical reliability methods rely

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on assumptions about the statistical distribution of variables which cannot be verified because of lack of data. Ferry Borges (ref 9) and Ang (ref 10) demonstrated the sensitivity to statistical distribution of variables of classical reliability techniques. Frendenthal (ref 11) advised that classical reliability theories will only rarely yield practical results.

Engineers inherently wish to design and build a structure that fulfils its intended purpose. It may be correct to say that most structures are a bit safer than they need to be to perform successfully because the cost of a modest degree of conservatism is small compared to the cost of failure. This influences discussions regarding the question of what constitutes acceptable reliability and appropriate safety factors.

An alternative to developing new safety factors is to conclude that existing practice has generally resulted in acceptable performance and to calibrate a new limit state standard against a prior working stress design (WSD) standard. This is an intellectually valid approach and probably a sensible course of action when developing a new standard because it:-

- Substantially reduces the amount of work required in developing a new standard.
- Reduces the scale of changes to industry practice both in new design and assessment of existing structures.
- Defers the work of reviewing and adjusting safety factors until there is industry experience in applying the new standard.

This approach was adopted in the development of API RP 2A LRFD which was calibrated against the 18th edition of RP2A WSD. The ISO 19900 series of standards has adopted the load factor (and implied reliability) from API RP2A LRFD for fixed steel structures but will evolve over time to have load factors appropriate to regional conditions.

The focus on reliability is generally seen to be on the safety factors and design equations in standards. A parallel but very important activity in developing a new standard, or a new edition of an existing standard, is the production of guidance that is easily understood and is likely to be applied in a consistent manner across the world with minimal scope for misunderstanding and errors. The impact on reliability of reducing the risk of errors in the application of a standard through clear unambiguous text is largely unquantified but is likely to be quite significant. A considerable effort has been made in the drafting of the ISO 19900 series standards to update the design guidance and to minimise the risk of incorrect application.

CROSS-INDUSTRY LIMIT STATE STRUCTURAL STANDARDS

Limit state general industry standards referenced by the oil industry have been developed by ISO, European Committee for Standardisation (CEN), Canadian Standards Association (CSA), American Concrete Institute (ACI), AISC, ASCE and Det Norsk Veritas (DNV). The general industry standards are highly important to the oil industry because they address aspects such as the detailed section design of topsides steel structure and of concrete structures. This allows the oil industry standards to focus on the material forms and loads unique to our industry and refer to globally available standards for all other aspects.

ACI, recognising that the allowable stress concept has limitations when applied to reinforced concrete design, were the first industry organisation to adopt limit state design principles. In 1956 they published 318-56 Building Code Requirements for Reinforced Concrete which introduced the ultimate strength methodology, which is conceptually similar to the LRFD methodology. The LRFD method has been used in ACI 318 since the 1970's. AISC published their LRFD Specification for Steel Buildings in 1986. In Europe the focus has been on developing a common set of limit state structural design standards for most applications and materials. This was completed in 2010 and comprises 58 separate documents addressing a wide range of materials. EN 1990 addresses the basis of design. EN 1991 address actions on structures. ENs 1992 to 1999 address steel, concrete, composite, timber, masonry, geotechnical, earthquake and aluminium.

OIL INDUSTRY DEVELOPMENT OF RELIABILITY BASED DESIGN STANDARDS

The development of limit state design structural standards used by the oil industry has taken place in a number of complimentary activities. Oil industry organisations have produced API RP 2A LRFD, Society of Naval Architects and Marine Engineers (SNAME) RP 5-5A, Norwegian (NORSOK) N-004, and the ISO 19900 series of offshore structures standards.

API commenced their work on an LRFD edition of RP2A in 1979 and published it for a two year review in 1989 followed by formal publication in 1992. The working stress edition has remained in place and is still more widely used than the LRFD edition. The work undertaken by API in the development of and calibration of API RP2A LRFD will be addressed by a separate paper in the conference.

In 1991 a study (ref 12) was conducted to identify the consequences for and adoption of API RP2A LRFD to North Sea Design Practice. The study comprised a Joint Industry Project undertaken by Advanced Mechanics and Engineering (AME) for 18 participants, including the oil majors. It set out to

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identify the partial factors for use in API RP2A LRFD that would provide a reliability when applied to North Sea conditions equivalent to a design undertaken to API RP2A WSD 18th Edition. It concluded that for extreme conditions (at that time the 50 year return event) an environmental factor or 1.3 could be applied compared to 1.35 in the GoM and a gravity load factor of 1.0 could be applied compared to 1.1 in the GoM. Using these modified factors for fully utilised components under in place extreme conditions a small weight penalty would arise compared to a design to WSD 18th Edition. For fully utilised components under gravity load a small weight reduction would arise. This study also investigated an alternative set of resistance factors for tubular joint design. A follow-on study identified factors for application to platforms in the Mediterranean.

An OTC 1992 paper (ref 13) documented the effect of applying API RP2A LRFD with North Sea partial factors derived from the AME study to the BP Andrew North Sea platform. This platform was designed to API RP2A 18th Edition WSD. It concluded that the overall effect on the structure weight is to increase it by a few percent. If however you take into account standardisation of member diameter and thickness to facilitate fabrication the variance in weight is minimal. It is thus possible to gain improved reliability with minimal cost consequence. The study did not address the topsides but it is likely that a potential saving of some 10% of the topsides weight could have been made by using LRFD rather than WSD which would have resulted in a worthwhile weight saving in the jacket and the lift installation process.

Faulkner and others at Glasgow University made extensive studies on the stability of cylindrical shells used for Semi-submersible and TLP design. These with related studies by Classification Societies and an Industry JIP led by Conoco resulted in limit state formulations for these key components. (ref 12 and 13).

DNV published their Classification Note 30.6 Structural Reliability Analysis of Marine Structures in 1992. Although intended for broad marine application this document has been primarily used by the oil Industry.

The SNAME Jack-Up Platform Assessment Recommended Practice 5-5A first edition was published in 1994. This practice introduced limit state design principles to the assessment of a jack-up.

Shell (ref 12) in 1996 undertook a study on reliability based criteria for fixed steel platforms. The method was based on evaluating the probability that the extreme environmental load will exceed the Reserve Strength Ratio (RSR) of a structure within a specified reference period. It included addressing local regional variations in the North Sea. The southern sector is sheltered from Atlantic seas. The northern sector is exposed to the Atlantic. The study showed the extent to which the limit state approach could be used by a an informed operator to tune load factors to obtain consistent acceptable reliability across a region.

In 2003 (ref 13) BOMEL on behalf of the UK Health and Safety Executive undertook a "System-based calibration for NW European waters for potential use in an ISO NW Europe Annex." This further developed the RSR related approach used in the prior work by Shell.

The Oil Industry ISO 19900 series standards were initiated because in the early 1990's the European Commission decided that it would develop standards to be applied to the oil industry. In response, to retain industry control over their standards, the oil industry decided that it would approach ISO with a proposal that the oil industry would themselves develop and maintain such standards under the ISO process. Many oil companies and oil industry contractors work internationally and have a direct interest in a single set of standards that can be applied worldwide. The European Commission has an agreement with ISO that they automatically adopt ISO standards as CEN standards. The process of developing the ISO 19900 series of offshore structures standards started in late 1991.

The early planning of the ISO standards considered the question of the adequacy of offshore structures individual and comparative performance, as understood in the early 1990's, and whether there was any requirement for amending the safety levels of the structures. All prior oil industry structural standards addressed only one structural material and form. The ISO standards are a suite addressing most types of fixed and floating platforms, jack-ups and arctic structures in steel or concrete. The industrial heritage of fixed steel platforms is largely unique to the oil industry. Concrete platforms utilise shell and column sections design principles that were developed for application to other structures. Floating structure monohulls have a heritage largely based on the shipping industry and column structures are generally based on a drilling heritage. All these have very different design and construction practice. At an early stage it was decided that it would be inappropriate to force a commonality of design and construction practice and reliability across the different types of structure and materials unless there are significant deficiencies.

A further consideration in the early planning of the ISO 19900 series of standards was the wish by many to avoid subdivision of the standard for a particular type of structure into multiple documents. The industry was comfortable with the format of the single document (API RP2A) used for fixed steel structures and in general did not wish to depart from this. The initial plan for the 19900 series was to have documents that apply across all types of structure for the general limit state design principles and the charaterisation of the marine environment and soils with a separate document for each type of structure. As the

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documents were developed it became obvious that further sub division was desirable to avoid the documents becoming unduly large, to minimise duplication and to facilitate revision. The format adopted has common sections applying general principles, load input data and foundations minimises the scope for errors and inconsistency across structural forms and materials, facilities revision and avoids unduly large volumes.

All reliability calculations need to be viewed with a degree of understanding of the state of the art. Engineers do not understand the statistics of extreme load events with sufficient accuracy to justify considering the reliability calculations as accurate figures. We are used to using code checks as a reasonably sound measure of the capability of a structure. We should not consider reliability calculations in the same light. For this reason the term notional reliability is often used which implies a value around which there could be quite a degree of variance.

INDUSTRY UNDERSTANDING OF THE RELIABILITY OF OUR STRUCTURES

The Industry has held two major prior meetings to address the issue of offshore structures reliability.

1. 1995

In 1995 BP sponsored a meeting to establish a baseline understanding of the reliability of each of principal forms of offshore structure. A group of 25 invited experts met at BP's UK training centre to identify the annual failure probability of the principal structural forms designed and built to best practice at that time. This meeting operated under an understanding that concluding that the experts could not conclude was not acceptable. The meeting was informed by a briefing paper prepared by Prof Moan and a day of presentations identifying the most relevant work on offshore structures reliability and the key limitations in the knowledge available at that time. Syndicates comprising academic and practicing engineers specialising in each structural form and material prepared an overview conclusion on the current (in 1995) annual failure probability of each type of structure. Both component and system failure were considered. The conclusions of each syndicate were reviewed in a common session to ensure consistency of approach and comparability of the conclusions. Each syndicate was then asked to further consider and refine their findings over a period of about one month. The findings were not for general publication. A report on the meeting was issued and made available to the meeting participants and experts drafting the ISO 19900series standards.

The principal conclusions of the 1995 meeting were:-

- Analysis procedures, methods and data have more potential to cause variability in reliability than the fundamental differences in structural form or material.
- There is broad comparability in reliability across structural form and material except for moorings and column structures air gap.
- Monohulls designed to trading vessel Class Rules do not have sufficient reliability for harsh environments and a supplement to these requirements is needed.

This conference occurred after and included work arising from the sinking of the Sleipner A concrete platform but pre dates the problems subsequently experienced with TLP tendon connections.

The first conclusion was an important influence in the decision by ISO TC 67/SC7 to avoid making design by reliability analyses an acceptable option unless unlikely exceptional circumstances apply. With the exception of the deficiencies identified in the second and third conclusion the meeting indicated to ISO TC 67/SC7 that the safety levels inherent in 1995 design best practice are acceptable.

A summary of the specific conclusion on the reliability of each type of structure is shown in figure 1. The low system reliability for concrete structures arose from limitations in global forces and shear capacity modelling identified in the SINTEFF investigation into the Sleipner A incident. This figure is not correct and did not influence ISO 19903 Concrete Structures.

2. 2012

The December 2012 Offshore Structures Reliability Conference was organised and sponsored by BP and held under the auspices of the Oil Industry Oil and Gas Producers Association (OGP) at Twickenham London. Attendance at his Conference was again by invitation only but compared to the 1995 event was on a much larger scale with more than 100 invited experts. Data from presentations was used to inform sessions in which participants updated the industry understanding of reliability. This conference included specialists interfacing with structural engineers including metocean, geotechnical, risers, seismic, moorings and drilling who were able to provide an insight on aspects of structural actions and component historic performance and predicted resistance. Reliability considerations for arctic structures, structures subject to seismic loading, marine operations and risers were addressed.

The conference substantially updated the industry understanding of offshore structures reliability compared to the

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1995 consensus view. It also broadened the range of structures for which best estimates of reliability were provided.
With the improved knowledge of reliability calculation process since 1995 it might be considered that the permissibility of design to a reliability figure might have become more acceptable. There was a strong view that this has not become a sound option and that the variability in the process of calculating reliability remains too great for it to be a sound routine design procedure. Notional reliability should continue to be used for the calibration of standards factors of safety rather than for bespoke design.

A report on this Conference was issued as OGP Publication 486 (ref 18). It included as Appendix B the report on the 1995 meeting previously issued only to participants and those involved in writing the ISO 19900 standards.

A table summarizing the findings of the 2012 conference is shown in figure 2.

FUTURE NEED FOR RELIABILITY CALCULATIONS

The work of people developing the general structural standards and the oil industry specific work undertaken primarily in support of API and ISO standards has advanced our understanding of the notional reliability of "dumb steel" structures to a point where we have reasonable confidence in the our figures. We have extended beyond dumb steel to include dynamic structures and moorings. We have a lower level of confidence in our understanding of the reliability of these structures. Our concern is not primarily with our knowledge of the steel components in these structures. It is with the totality of the system including the human factors and controls associated with mechanical systems critical to the integrity and operability of the structure. Our challenge will be to combine our structural reliability calculations with human factors and software controls to derive a holistic understanding of system reliability.

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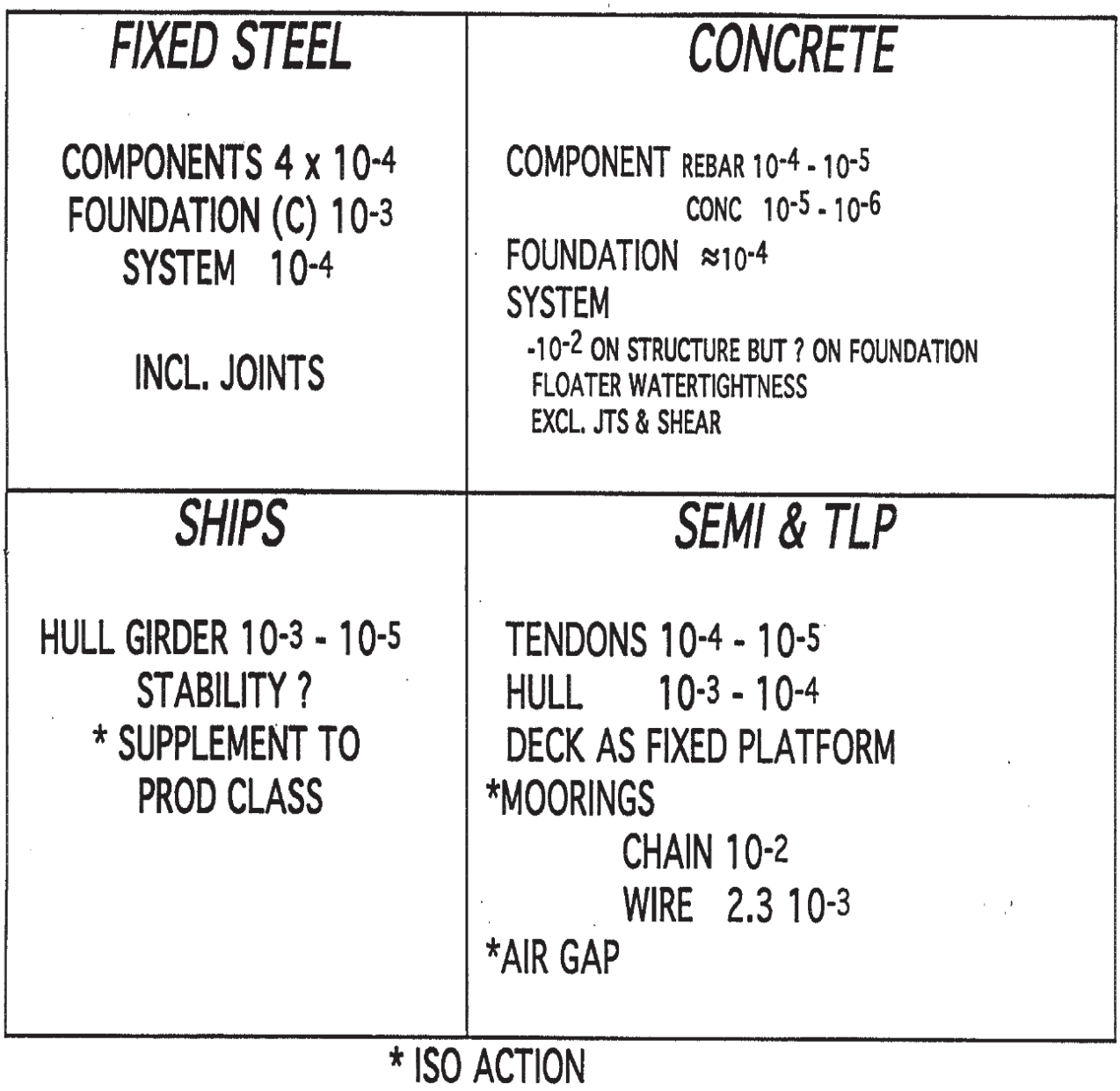
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FIGURE 1 1995 RELIABILITY CONFERENCE SUMMARY CONCLUSIONS



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FIGURE 2 2012 RELIABILITY CONFERENCE SUMMARY CONCLUSIONS

NOTIONAL ESTIMATED FOR PER ANNUM PROBABILITY OF FAILURE
EXPOSURE LEVEL L1 - OTHER PLATFORM TYPES (AS CURRENTLY DEFINED IN ISO)

Non-cyclonic region— Permanently manned	**Cyclonic region—evacuated— US Gulf of Mexico**	**Other regions/comments**
Ultimate Limit State—ULS		
First component ~1×10^{-4}	First component ~4×10^{-4}	ISO member & joint checks typically 10–20% higher than API. Other regions likely to lie between 1×10^{-4} and 4×10^{-4} depending upon which Standard applied.
n.b. Appurtenances in wave action zone including caissons, boat impact framing, etc are liable to fail at higher probability levels.		
Collapse ~3×10^{-5}	Collapse <1×10^{-4}	
Accidental/Abnormal Limit State—ALS (non-ice)		
Collapse* <3×10^{-5} Hazard curve low slope so ULS dominated	Collapse* ~5×10^{-4} 2000 year return period Non-robust design, *e.g.* K braced	*Provided no wave on deck due to old/poor air gap design or platform subsidence Other regions: Collapse* <1/ALS return period. ALS design check based on safety factors of 1.0 applied to some characteristic variables.
	Collapse* ~1×10^{-4} Robust design, *e.g.* X braced [for context consult Annex B] Hazard curve high slope so ALS dominated	
Accidental/Abnormal Limit State—ALS (Arctic Ice)		
Collapse ~1×10^{-5} Collapse <1×10^{-4}	Not applicable in cyclonic regions	Factors of safety in ISO 19906 specifically derived to achieve these specified probability of failure levels.
Foundations		
Modest lower probability of failure in foundation than jacket is predicted, provided site specific soil data has been employed in design		Site specific means boring data obtained at a location in close proximity to the installation. In addition, access to pile driving records can reduce uncertainty in foundation probability of failure.

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NOTIONAL ESTIMATED FOR PER ANNUM PROBABILITY OF FAILURE
EXPOSURE LEVEL L1 - OTHER PLATFORM TYPES (AS CURRENTLY DEFINED IN ISO)

Seismic	
Seismic Region 4: Collapse $<4 \times 10^{-4}$ Collapse ~1.9 × 10-4 to 2.5 × 10-4 (*e.g.* Pacific rim, Indonesia)	ALE=2,500yr return period event, but typically this relates to probability of around 1/4000 yr to 1/5300 yr
Seismic Region 3: Collapse ~1.5×10^{-4} (*e.g.* West Indies, Turkey) [by interpolation]	
Seismic Region 0 to 2: Collapse $<1 \times 10^{-4}$ (*e.g.* GoM, North Sea, Brazil, West Africa, Australia) [For context consult section 6 and related presentations]	
Fatigue Limit State—FLS	
Despite good intention there was little data presented on component or system degradation leading to failure (except for mooring systems). Fatigue cracks can generally be managed, while more severe loss of connectivity is perceived to be relatively rare. The exceptions being in the wave action zone, *i.e.* conductor guide frame, appurtenances, caissons, clamps, boat impact framing, *etc*, failure of which was perceived as not uncommon.	Normative and informative tables with fatigue safety factors in the range 2.0 to 10.0 are creeping into Standards. It was perceived that these were probably derived in a manner consistent with other structural risks. However, applying these factors to generally very conservative fatigue calculation methodologies may lead to probabilities of failure for primary members and joints well below other limit states.
Fixed concrete structures	**Comments**
First Component: Rebar ~10^{-4} to 10^{-5} Concrete ~10^{-5} to 10^{-6} Foundation ~1×10^{-4} System failure is perceived as being two orders of magnitude lower, *i.e.* System collapse ~1×10^{-6}	Concrete structures were not discussed at this conference and therefore the notional reliability values from 1995 have been reproduced unchanged.
Jack-ups	**Comments**
Foundation Collapse $<1 \times 10^{-3}$ Structural Collapse similar magnitude to that for non-robust configuration fixed steel jacket structure.	
Floating Production Units and MOU—Vessel	**Comments**
Offshore structure based on Class Rules. Need Classification Society advice on perceived reliability if vessel designed and operated to Class. A notional component strength 10^{-3} was quoted in the absence of any new data. *n.b. component failure rates in range 10^{-3} to 10^{-4} were estimated for production semis and TLPs in 1995 conference (Annex B).* Hull girder perceived as low probability of failure. Fatigue and corrosion are significant issues that need to be appropriately managed.	Does seismic risk for production platforms need to be considered too?

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NOTIONAL ESTIMATED FOR PER ANNUM PROBABILITY OF FAILURE
EXPOSURE LEVEL L1 - OTHER PLATFORM TYPES (AS CURRENTLY DEFINED IN ISO)

Floating Production Units and MOU—moorings	Comments
MOU mobile moorings—strength: 10yr criteria: One line failure ~7×10^{-2} System failure ~4×10^{-2} 25yr criteria: One line failure ~1×10^{-2} System failure ~7×10^{-3} 50yr criteria: One line failure ~4×10^{-3} System failure ~2×10^{-3} 100yr criteria: One line failure ~ 2×10^{-4} System failure ~9×10^{-5} (GoM API RP 2SK study)	Too many MOU mooring failures are perceived to be occurring. However increasing the design safety factors and/or return period design criteria in isolation may not improve reliability to the level predicted.
MOU mobile moorings—fatigue and anchoring: No data presented.	
Production permanent moorings—strength: DNV Posmoor: Failure ~10^{-4}	Further work is required to establish if this value relates to first failure or system failure, and also to locate similar probabilities in API RP 2SK/ISO 19901-7
Production permanent moorings—fatigue: SN DNV & ISO inf.: One line failure ~10^{-3} System failure ~ 10^{-5} TN API & ISO norm.: One line failure ~1×10^{-4} Actual failure rate data reported (over last decade): One line failure ~2×10^{-2} System failure ~3×10^{-3} *(n.b. majority of mooring line failures are non-design,* i.e. *poor construction, damage during installation, or inappropriate operation).*	The SN fatigue approach is based on DNV Deepmoor calibration in 1998, while the TN fatigue approach was verified by API RP 2SK Work Group in March 2003. ISO 19901-7 recommends the TN approach in its normative text but alternatively permits the SN approach in the informative annex. In some cases, higher factors of safety have been perceived to add complexity and risk to fabrication and handling.
Production permanent moorings—anchoring Few, if any, failures of anchoring systems reported. Perceived to be low probability of failure but not sufficiently quantified to warrant reduction in safety factors.	
TLP tendons: Not specifically addressed, but 1995 conference considered a component failure probability of 10-4 to 10-5 This reliability level is now perceived to be possibly optimistic.	
Floating Production Units and MOU—risers	**Comments**
Very limited data around riser reliability. Leak failure rates in the range 5×10^{-3} to 1×10^{-4} per riser per year are suggested to initiate further discussion and studies	A desire to integrate the riser community into the offshore structure community's risk and reliability assessments was encouraged.

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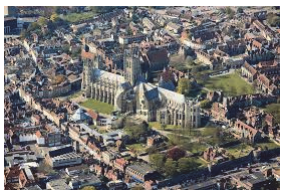
History behind the development of Limit State (LRFD) Offshore Structures Design Standards

Richard Snell

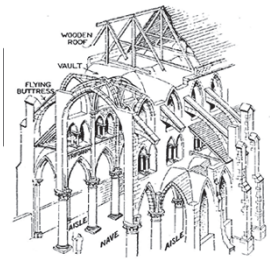
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Historical applications of standardisation

- Military - fast build, limited life
- Religious & mausoleum
  - Pyramids
  - Cathedrals







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Pioneers of the principle of consistent design – before the concept of reliability evolved

- 1801 Attwood Disertation on the Construction and Properties of Arches
- 1826 Navire Resume des Lecons de Mechanique
- 1840 Hodgkinson On the strength of pillars of cast iron and other materials
- 1864 Fairburn On the applications of cast and wrought iron to building purposes

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Safety Factors in a standard

- Allowable or Working Stress
  - –Single factor accounts for all variables
  - –The norm until 1980's
- Limit State (LRFD)
  - –Enable design safety factors to reflect the local environment and function of the structure.
  - –Provide consistent reliability in structures.
  - –Without the designer having to undertake a reliability analysis as part of the design process.
  - –Adopted by many industry associations and standards bodies from 1980's.

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Evolution of reliability based design principles

- 1936 AM Freundenthal Columbia University working on reliability in civil engineering structures. In 1947 published “The safety of structures”.
- 1942 AG Pugsley Bristol University working on reliability in aircraft structures. In 1951 published “Concepts of safety in structural engineering”.
- Both were of the view that structural engineers knowledge of structural reliability lags far behind their ability to undertake complex analysis.
- Both viewed the key need to be improved statistical understanding of the loads to which structures are subject.
- 1956 Frendenthal seminal paper “Safety and the probability of structural failure” provided definitions for unserviceability and failure and outlined procedures for the numerical evaluation factors of safety.

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How we express structural reliability

- Imperfectly
  - Not measured by mechanical or function tests against a precisely defined duty.
  - Scale of our structures and the magnitude of loads makes that impractical.
  - We predict reliability based on a defined design and construction process and the statistics of load and material resistance.

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Historical evidence as a means of quantifying reliability

- Identifies the obvious but not good at subtle trends due to changes
  - material production processes
  - performance and consistency of materials
  - sampling, quality control and inspection methods
  - site assembly technique
  - dimensional accuracy
  - materials handling
  - documentation and records
  - service requirements
  - Frendenthal in 1968, Ang & Amin in 1969 and Ferry Borgas and Castanheta in 1971 all concluded that classical reliability theories will only rarely yield practical results.

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Calibration against an existing standard

- A valid approach when developing a new standard is to calibrate it against a prior standard deemed to have delivered acceptable performance.
- This substantially reduces the work and time involved in delivering a new standard, reduces the scale of change imposed on industry and defers calibration until experience of the new standard has ironed out significant deficiencies.
- This approach has been used for API RP 2A-LRFD which was calibrated against the 18th ed. WSD and in ISO 19902.

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General Industry Limit State Standards

- The oil industry is rather late in adopting limit state structural standards.
  - ACI in 1956 published 318-56 Building Code Requirements for Reinforced Concrete. This was conceptually similar to LRFD and since 1970 have used an LRFD based version of 318.
  - AISC in 1986 published their LRFD Specification for Steel Buildings.

  - CEN in 2010 completed their 58 part EN 1990 series standards addressing steel, concrete, composite, timber, masonry, geotechnical, earthquake and aluminium.

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First major oil industry application - API RP 2A

- API commenced work on LRFD in 1979 and first published it for a 2 year review in 1989.
- Supported by a major review of the GoM marine environment and a structural member and joint calibration study.
- Was the principal oil industry limit state standard for some 25 years and is the base document for the development of ISO 19902.
- More detail on the development of API RP 2A is the subject of a separate talk.

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Industry studies related to API RP 2A

- 1991 AME JIP calibrated factors for LRFD to attain similar reliability to applying 18th ed. WSD to the N Sea.
- 1992 OTC paper 6931 recorded an internal BP study on the effect of applying LRFD with the AME factors to a North Sea platform design.
- Shell in 1996 addressed consistency of reliability across the North Sea by evaluating the probability that the extreme environmental load would exceed the RSR within a specified reference period.
- In 2003 BOMEL applied the Shell approach to evaluate partial factors for the first edition of the ISO Fixed Steel Platform Standard (RP 2A LRFD under an ISO wrapper) for NW Europe.

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Other Industry reliability based work

- Faulkner et al at Glasgow University studied the stability of cylindrical shells for column stabilised structures. Led to an industry JIP developing limit state formulations for columns
- 1994 SNAME Jack-Up Platform Assessment Recommended Practice first edition published which applied limit state principles to the assessment of a dynamic structure.
- 1992 DNV Classification Note 30.6 Structural Reliability Analysis for Marine Structures published.

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Consistent reliability is more than equations and factors

- Our focus in terms of reliability tends to be on the design equations and partial factors.
- Clarity of guidance in a standard is also extremely important.
- Need to avoid errors arising from misunderstanding, ambiguous language and inconsistencies in standards.
- Need to minimise scope for adventurous interpretation.
- It is not a trivial task to produce clear and unambiguous guidance and is one of the key reasons why it takes a long time to write a new standard.

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Industry current best estimate of the reliability of offshore structures

- The two prior OSRC's took the output of the component and system calibrations and studies available at that time and debated and summarised our best understanding.
  - In 1995 for informing work on the ISO Offshore Structures Standard.
  - In 2012 updating and significantly broadening the scope of the 1995 work.

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Summary of 1995 conference

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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The principal conclusions of the 1995 meeting

- Analysis procedures, methods and data have more potential to cause variability in reliability than the fundamental differences in structural form or material.
- There is broad comparability in reliability across structural form and material except for moorings and column structures air gap.
- Monohulls designed to trading vessel Class Rules do not have sufficient reliability for harsh environments and a supplement to these requirements is needed.

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2012 Conference

Fixed Steel

Component $4x10^{-4}$

Collapse $3x10^{-5}$ / $<10^{-4}$

ALS $<3x10^{-5}$ / 1 to $5x10^{-4}$

Concrete

Rebar 10^{-4} to 10^{-5}

Concrete 10^{-5} to 10^{-6}

Foundation 10^{-4}

System 10^{-6}

FPU and MOU

Component 10^{-3} to 10^{-4}

Component to Class Rules 10^{-3}

FPU and MOU Moorings

Strength $2x10^{-4}$

System $9x10^{-5}$

Fatigue one line 10^{-4}

Actual failure rate over last decade

One line $2x10^{-2}$

System $3x10^{-3}$

Jack-Up

Structure & Foundation 10^{-3}

Riser

Leak rate per riser per year $5x10^{-3}$ to 10^{-4}

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Principal technical conclusions of the 2012 meeting

- Design by reliability analysis is no more valid than at the time of the 1995 conference. Notional reliability should continue to be used for the calibration of standards factors of safety rather than bespoke design.
- Stationkeeping standards bodies to seek improved reliability in mooring systems and thruster assisted moorings by sharing lessons from failures and reliability based assessment studies from both MOU and production platform communities.
- Stationkeeping standards bodies, Classification Societies and operators to seek to converge their respective requirements for stationkeeping systems.
- Riser community to seek ways to share information on riser incidents and associated reliability of riser systems, possibly via OGP Structures Subcommittee which now includes riser systems in its Terms of Reference.

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Notional Reliability

- Our predicted reliability figures are not exact numbers.
- We use the term “Notional Reliability” to emphasise that they need to be considered with appropriate caution.
- Most of our big accidents have arisen from causes that are not uniquely structural engineering.
- Floating structures in particular bring major risks from ballast system errors, compartment inspection, stationkeeping and risers.

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Major Accidents

- Accidents involving large loss of life
  - 1965 Sea Gem
  - 1974 Gemini
  - 1976 Ocean Express
  - 1980 Alexander Kielland
  - 1882 Ocean Ranger
  - 1988 Piper Alpha
  - 2005 Bombay High

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No loss of life but wake up Accidents

- 1991 Sliepner A Concrete
  - FE analysis and rebar detail
- 2002 P34 FPSO
  - Ballast system control
- 2005 Thunder Horse semi
  - Ballast control piping
- 2005 Typhoon 3 leg TLP
  - Tendon system
- 2006 Dry transport vessel sinking
  - Valve failure

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Reliability considerations in 2014

- We have developed a reasonable notional understanding of the reliability of “dumb steel” oilfield structures. We are reasonably confident that our standards, correctly applied, should avoid a major design or fabrication error in the structure.
- We are increasingly using structures that are a combination of dumb steel and active systems.
- In this more complex world human factors and control systems operating mechanical equipment fundamental to structural integrity and operability, especially stability, are a major component of system reliability.

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Basic Principles of Structural Reliability

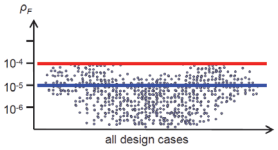
Adel Younan
ExxonMobil Upstream Research Company

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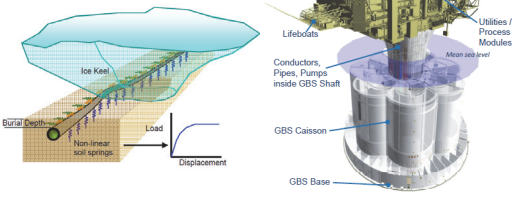
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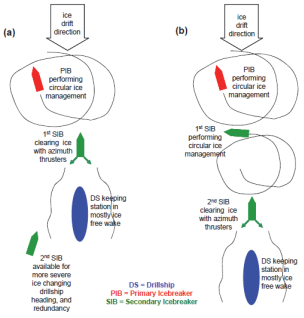
Why & When?

- Basis for code calibration
  - API RP2A LRDF
  - ISO 19906
- Limit states not covered by code:
  - P/L burial depth for ice gouging)
  - Iceberg loads on structures
  - Ice management reliability
- Structural Reassessment
  - Fit-for-purpose designs
  - Life extension of facitlities



Source - Calibration of action factors for ISO 19906 Arctic offshore structures, OGP Report 422





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WHY – *The Driver*

It ain't what you don't know that gets you into trouble
It's what you know for sure that just ain't so - Mark Twain

1. **Design Uncertainty**

2. **Societal Expectations**

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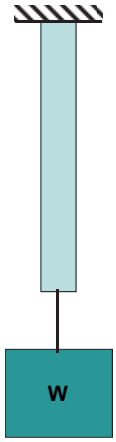
1. DESIGN UNCERTAINTY

Example 1

- Yield strength f_y = 60 ksi
- Weight, W = 60 kips.
- Find A = W / f_y. Estimate probability of failure.
- Depending on uncertainty consideration, $P_{failure}$ could range between 60% and 1/10000.

Unaware

	Tom	Jim	Joe
f_y (ksi)	60 ± ?	60 ± 10	60 ± 10
W (kips)	60 ± ?	60 ± ?	60 ± 24
A (in^2)	1	1.2	1.67
$P_{failure}$	0.60	0.25	0.00014



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2. SOCIETAL EXPECTATIONS

Definitions

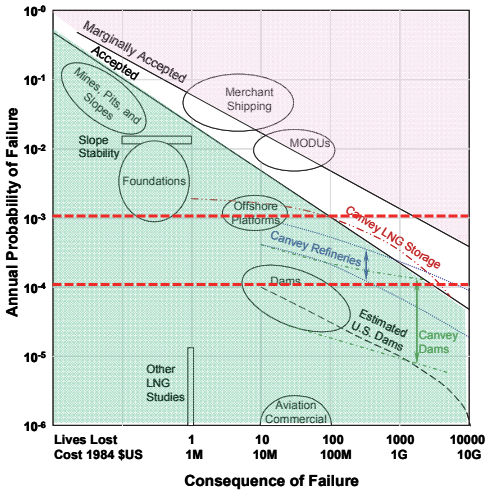
- Hazard = Probability (Likelihood) of failure
  = 0 to 1
- Risk = Likelihood * Consequence
  = Monetary value ($) , qualitatively categorized between small up to simply unacceptable
- Annual risk
  - risk per year, industry standard
  - used in setting and comparing safety goals
- Project life risk
  - Controversial, historical: **longer** life project means **higher** design factors (and less annual risk)
  - Should avoid project life risk for derivation of criteria
- Actuarial risk
  - Based on actual failure data
  - Used for mass produced units (gadgets), **difficult** to quantify for civil/offshore structures and **impossible** to quantify for first time application
- Notional risk
  - Based on numerical estimation of risk (hopefully calibrated against real data)
  - Used to set safety goals and developing design factors

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2. SOCIETAL EXPECTATIONS – *Contd.*

Safety Goals

- Annual probabilities of failure implicitly accepted by society over the years:
  - Actuarial data
  - Strong correlation with consequence
- Notional reliabilities
  - Used for performance goals and establishing design factors
  - Big variation between industries
    - Nuclear: 10^{-3} to 10^{-4}
    - Oil: 10^{-4} for major failure
  - Can vary with likelihood and consequence, i.e. risk-based (US DOD risk matrix)
  - Can be based on cost-benefit analysis (cost of life) -- may cause controversy and legal issues in US



Source: Whitman and Baecher, 1985.

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2. SOCIETAL EXPECTATIONS – Contd.

Likelihood of Hazard

MILITARY STANDARD RISK MATRIX
From USA DOD MILITARY STD 882c, 19 Jan. 1993, p. A5

Consequence of Hazard

Annual Frequency	Hazard Category (1) Catastrophic	(2) Critical	(3) Marginal	(4) Negligible
(A) Frequent ($x > 10^{-1}$)*	1A	2A	3A	4A
(B) Probable ($10^{-1} > x > 10^{-2}$)*	1B	2B	3B	4B
(C) Occasional ($10^{-2} > x > 10^{-3}$)*	1C	2C	3C	4C
(D) Remote ($10^{-3} > x > 10^{-6}$)*	1D	2D	3D	4D
(E) Improbable ($10^{-6} > X$)*	1E	2E	3E	4E

* Example of quantitative criteria

Hazard Risk Index	Suggested Criteria
1A, 1B, 1C, 2A, 2B, 3A	Unacceptable
1D, 2C, 2D, 3B, 3C	Undesirable (MA decision required)
1E, 2E, 3D, 3E, 4A, 4B	Acceptable with review by MA
4C, 4D, 4E	Acceptable without review

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HOW – *The Engine*

It is better to be approximately right than precisely wrong – Warren Buffet

4. **Probability Refresher**
5. **Limit States**
6. **Reliability Basics**
7. **Fixed Platform Reliability**
8. **Design Formats**
9. **Types of Errors**
10. **Inspection Intervals**
11. **Interpretation of Results**

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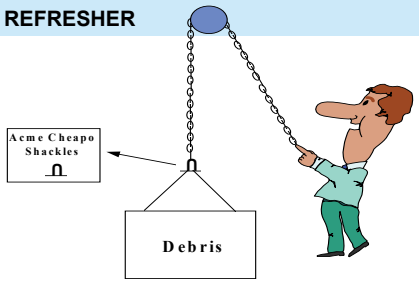
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4. PROBABILITY REFRESHER

Example 2

- Use a shackle to lift a box of debris.
- Select shackle from 3 "identical" ones.
- Shackle breaking strength = 5000, 7500, or 10,000 lbs.
- Box weight = 4000, 6000 lbs, or 8000 lbs. These values are equally probable.
- What is the probability that the debris box will break loose from the chain fall?
- 9 possible outcomes as listed in table.
- $P_{failure}$ = P [strength < box weight]
  = 3 failures / 9 possible outcomes
  = 0.33
- $P_{success}$ = Reliability
  = 1 – $P_{failure}$
  = 0.67



Outcome	Shackle Strength	Debris Weight	Outcome
1	5000 lb	4000 lb	OK
2	5000 lb	6000 lb	Fail
3	5000 lb	8000 lb	Fail
4	7500 lb	4000 lb	OK
5	7500 lb	6000 lb	OK
6	7500 lb	8000 lb	Fail
7	10000 lb	4000 lb	OK
8	10000 lb	6000 lb	OK
9	10000 lb	8000 lb	OK

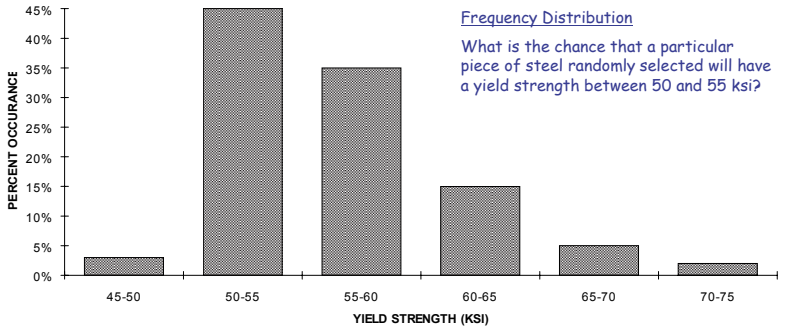
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4. PROBABILITY REFRESHER – Contd.

Random Variables

- Very few quantities are deterministic (i.e., not random).
- Quantifying uncertainty requires mathematical models of "random variables".
- For e.g., the yield strength of steel is a random variable. When we buy steel, a range of strengths are possible, but we specify a nominal value when ordering.
- To get a handle on possible strength, we could test the steel and establish frequency distribution



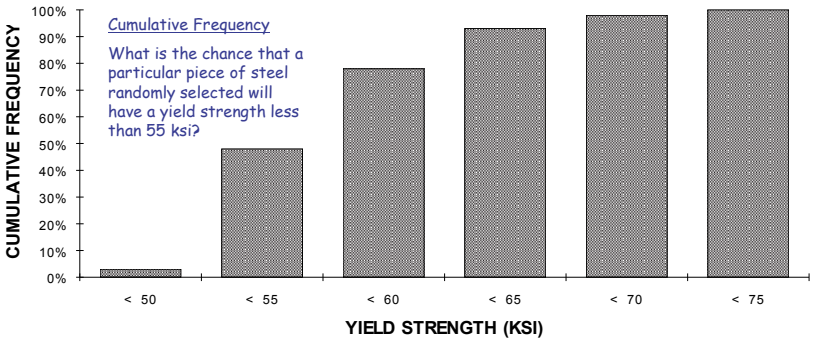
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4. PROBABILITY REFRESHER – Contd.

- We need a sufficient number of samples to provide statistically significant answers.
- Frequency distribution → Cumulative distribution → Mathematical models for random variable.
- RVs are usually continuous rather than discrete.
- Statistical Inference = estimating parameter and distributions from observational data.
- Need to be cognizant of any bias in sample; e.g., difference between in-lab and in-field welds.

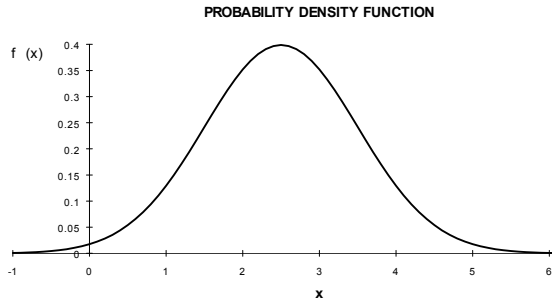


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4. PROBABILITY REFRESHER – Contd.

Probability Density Function

- Continuous RV, X.
- $f_X(x)$ = PDF of X
  - P [$a \le X \le b$] = Area under curve between a & b.
  - P [$a \le X \le a$] = 0
- $f_X(x)$, must satisfy two rules
  - $f_X(x) \ge 0$ for all x
  - Total area under $f_X(x)$ curve must be equal to 1.0.
- $f_X(x_1, x_2, ...)$ = multivariate PDF
  - Captures randomness of and dependence between several RVs.
  - More complex, but still has to follow these two simple rules.



$$P[a \le X \le b] = \int_a^b f_X(x)dx$$

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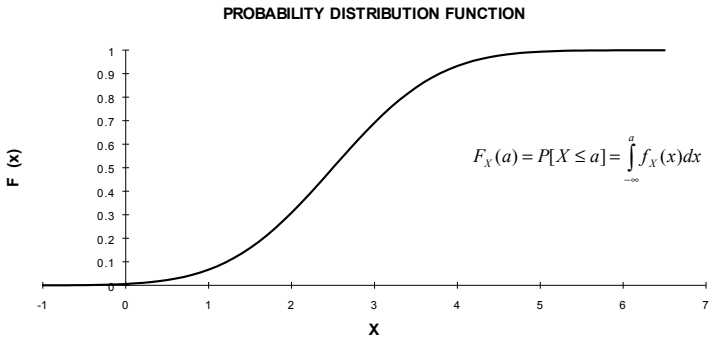
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4. PROBABILITY REFRESHER – Contd.

Probability Distribution Function

- Sometimes known as the cumulative distribution function CDF or a non-exceedence curve.
- $F_X(x)$ = area under $f_X(x)$ function from lower limit to x
- The bounds on $F_X(x)$ are 0 and 1.



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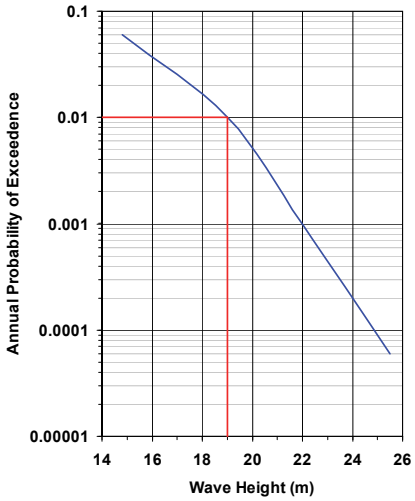
4. PROBABILITY REFRESHER – Contd.

CCDF & Hazard Curve

- We are often interested in estimating the probability of exceeding a given value

  $1 - F_X(x)$

- Exceedence = complementary of CDF
  = CCDF
- Hazard Curve = annual exceedence
  = annual CCDF
- GOM wave height hazard curve
  Max wave, W = max wave height in a storm
  Hazard curve = annual exceedence of max wave height in a storm
  - P [$W \ge 19$] = 1% = 1/100
  - W_{100} = 19 m.
- $W_{100} \neq$ max storm wave in 100 years
  W_{100} = max storm wave with 1% chance of exceedence in a single year



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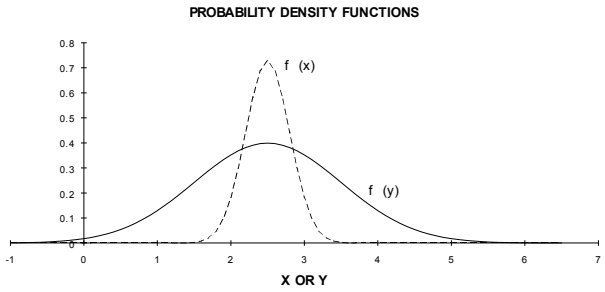
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4. PROBABILITY REFRESHER – Contd.

Moments of a Random Variable

- Two standard characteristics of a random variable:
  - Central tendency (mean or average, median, mode)
  - Dispersion (standard deviation, rms, variance, coefficient of variation (COV)).
- Typical dispersion values:
  - Steel yield strength typically has a COV of 5%
  - Fatigue endurance typically have COV's of greater than 50%.



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4. PROBABILITY REFRESHER – Contd.

Common Probability Distributions Used in Offshore Engineering

- Normal (Gaussian) Distribution
  - Spans parameter space from -∞ to +∞
  - Physically used to model the sea surface elevation
  - Commonly (often incorrectly) used for RVs where we have no distribution information
- Rayleigh Distribution
  - Spans parameter space from 0 to + ∞
  - Peaks of a narrowband, Gaussian process are Rayleigh distributed – e.g. max wave heights
- LogNormal Distribution
  - Spans parameter space from 0 to +∞
  - Many physically measured quantities – steel yield strength, S-N data, etc ...
- Extreme Value Distribution
  - Can mathematically derive distributions to model the maximum (or minimum) of a RV
  - This family of distributions is used to model extreme wave heights, etc...

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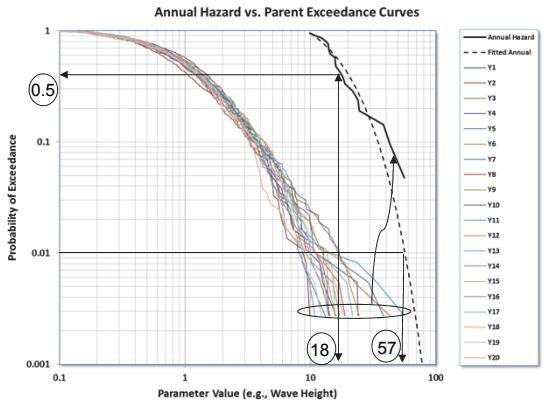
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4. PROBABILITY REFRESHER – *Contd.*

Extreme Value Analysis

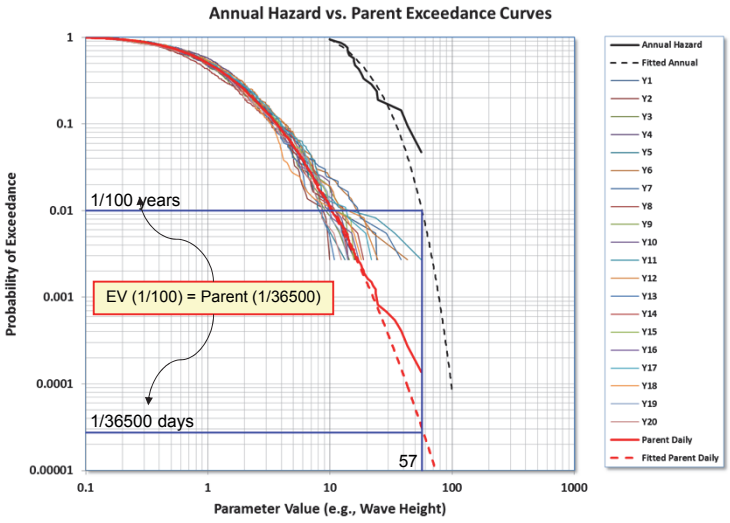
- Parent Distribution – daily data for 20 years
- Extreme Value Distribution – annual maxima
- Parent = Daily Exceedance
- Annual Hazard = Exceedance curve of EV Distribution
  - Structure capacity may be good for 364 days, but the 1 day it is exceeded (365th) is what counts
- Worthy of notice:
  - Parent (.01) exceedance = 18; EV (.01, 100-year) = 57
  - Parent (0.01) of (18) corresponds to EV (0.5, 2-year)



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4. PROBABILITY REFRESHER – *Contd.*



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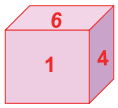
4. PROBABILITY REFRESHER – Contd.

Classification of Uncertainty

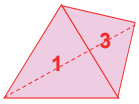
Example 3

- Unknown dice with 4 rolling outcomes: 1, 3, 3, 5
- Task: Develop a model for unknown dice
  - Model 1 - cubical dice with six sides: 1 to 6
  - Model 2 - 4-sided pyramid with two 3's: 1, 3, 3, 5
  - Model 3 - 4-sided pyramid: 1, 3, 5, 4(?)
  - Other models ...
- Which is correct? Need additional data.
- So for now, we should consider possible models:
  - As equally credible? Or
  - Can we weight them based on "expertise"
- Need to distinguish between two types of uncertainty:
  - Randomness, Aleatory or Type I
  - Modeling, Epistemic or Type II

Side	Probability			
	Model 1	Model 2	Model 3	...
1	1/6	1/4	1/4	?
2	1/6			?
3	1/6	1/2	1/4	?
4	1/6		1/4	?
5	1/6	1/4	1/4	?
6	1/6			?



Model 1



Model 2, 3

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4. PROBABILITY REFRESHER – Contd.

Type I Uncertainty

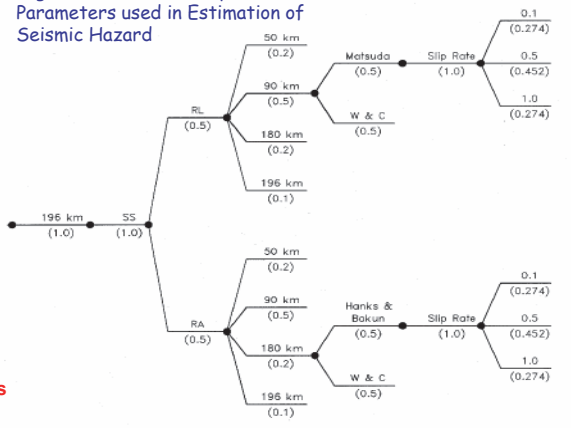
- Natural randomness - wave heights, wind speeds, yield strength, earthquake intensity
- It is not reduced with more data

Type II Uncertainty

- Due to lack of knowledge
- Can be reduced, even eliminated with more data
- Examples
  - Morison's equations
  - EQ fault rupture modeling
  - Failure mode for an ice floe
- Incorporated in analysis, for e.g., through logic trees.
- Type II uncertainty many times dominates, but **unfortunately is often neglected**.

Total Length	*Sense of Slip*	*Maximum Magnitude Approach*	*Characteristic Rupture Length*	*Empirical Relationship*	*Recurrence Approach*	*Slip Rates (mm/yr)*

Logic Tree for Fault Rupture Parameters used in Estimation of Seismic Hazard



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4. LIMIT STATES

- Limit State (Z) = Description of a specific failure mode.
  Limit states for "failure modes" of an offshore platform might be:
  - Diagonal brace buckling
  - Pile pull out
  - Joint fatigue
  - Jacket collapse
- Reliability results must be considered in the context of limit states. Saying "the probability of failure of this platform is 10^{-3}" is meaningless without stating failure modes considered.
- In general terms, we can define a limit state, Z, as:

  Z = Resistance - Load

  We then define failure as when $Z \le 0$. This implies:
  - Failure $Z \le 0 \Rightarrow$ Resistance ≤ Load
  - Reliability $Z \ge 0 \Rightarrow$ Resistance ≥ Load
- This allows a wide interpretation of load and resistance. For example, for a joint fracture problem, the "resistance" could be the critical flaw size and the "load" would then be the calculated crack size.

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4. LIMIT STATES – Contd.

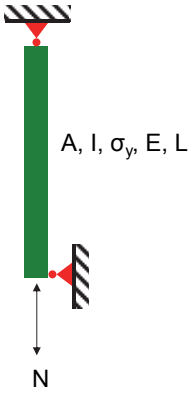
- With the above definition for the limit state, the probability of failure or success (reliability) are then:
  - Prob. of Failure = P [Z ≤ 0] = P [R ≤ L]
  - Reliability = P [Z ≥ 0] = P [R ≥ L] = 1 - P [R ≤ L]

  (R = Resistance; L = Load)

Example 4

- For this steel bar loaded axially, what are some possible limit states?

Limit state	Limit State Function	Reliability
Tension	$Z = A\,\sigma_y - N$	$P[\,A\,\sigma_y \ge N\,]$
Buckling	$Z = \frac{\pi^2 EI}{L^2} - N$	$P[\,\frac{\pi^2 EI}{L^2} \ge N\,]$
Deflection	$Z = \Delta_{ALLOW} - \frac{NL}{AE}$	$P[\,\Delta_{ALLOW} \ge \frac{NL}{AE}\,]$



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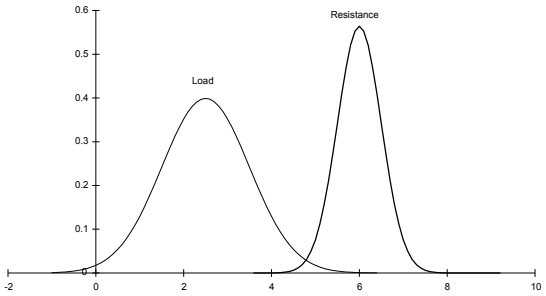
4. RELIABILITY BASICS

Engineering Meaning of Reliability

- Reliability = Probability of performance at or above a given standard. (0 to 1 - 0 is least reliable)
- Probability of Failure = Probability of performance below a given standard. (0 to 1 - 1 is certain failure)
- Since these two events (success and failure) are complements, the following is true:

  Reliability = 1.0 – Probability of Failure

  $p_s = 1.0 - p_f$
- Typically, we calculate the probability of failure and then find the reliability.



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5. RELIABILITY BASICS– Contd.

Approach

Doing a reliability analysis boils down to 3 steps:

1. **Define the limit state**
   Sometimes this is the hardest part. What do you want to analyze? What engineering assumptions are acceptable? Can you define the failure region analytically?
2. **Define the random variables**
   Which variables will be assumed random? What are the distributions? What are the statistics of the random variables?
3. **Calculate the probability of being in the safe region**
   As defined by the limit state:

$$\text{Reliability} = P[Z \geq 0] = \int_{\text{Safety Region}} f_z(z)dz$$

or alternately,

$$\text{Reliability} = 1 - \text{Probability of Failure} = 1 - P[Z \leq 0] = 1 - \int_{\text{Failure Region}} f_z(z)dz$$

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5. RELIABILITY BASICS – Contd.

Methods of Analysis

1. ***Direct Integration***
   - Direct integration of probability density function (see equations in previous slide).
   - Requires explicit limit state function & an analytical probability density function.
2. ***Monte Carlo Simulation***
   - Step 1 - "Randomly" simulate one realization of RVs that define the limit state.
   - Step 2 - For this realization of RVs, calculate the limit state function.
   - Step 3 - If the limit state is less than 0 (failure occurs), add 1 to the total failures counted.
   - Step 4 - Go back to Step 1 (unless you have enough experiments)
   - Step 5 - Add number of failures counted in step 3 and divide by the number of simulations.
3. ***First/Second Order Reliability Method (FORM / SORM)***
   - First, convert RVs into standard normal variables and limit state into a new "g" function.
   - Determine shortest distance from the origin to the g function.
   - The probability of failure is found to be $p_f = \Phi(-\beta)$

Reliability Index	0.0	1.0	2.0	3.0	4.0	5.0
Notional P_f	0.5	0.16	0.023	1.40E-03	3.20E-05	2.90E-07

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6. FIXED PLATFORM RELIABILITY

1. Limit State Definition

- Probability of "collapse" in a storm -- failure occurs when load > platform resistance
- Failure is typically described by
  - excessive deck displacement or
  - onset of instability (pile pull-out, etc...)

2. Random Variable Definition

- Load random variables:
  - Wave height (H)
  - Storm frequency (λ)
  - Current, wind, direction
  - Wave calculation uncertainty (ε)
- Resistance random variables:
  - Material strength (low COV)
  - Joint strength (age of platform)
  - Damage, corrosion
  - Modeling of member behavior
  - Modeling of soil and foundation

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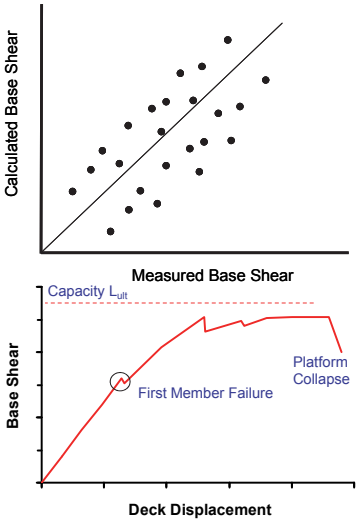
6. FIXED PLATFORM RELIABILITY – Contd.

3. Calculation of Reliability

- ***Fully nonlinear analysis***
  - Monte Carlo simulation - sufficient number of experiments to simulate platforms subjected to random storms.
  - FORM/SORM - By coupling a FORM/SORM solver with a nonlinear structural analysis code
- ***Simplified analysis***
  - Wave height dominates environmental uncertainty
  - Resistance uncertainty is negligible
  - Express global platform load as a function of wave height ˜from series of static analyses
  - Nonlinear structural analysis to obtain global load to cause failure, L_{ult} (assume uniqueness)
  - Calculate the probability of failure:

$$P[\, L(H, \varepsilon, \lambda) \geq L_{ult} \,]$$

- Typical annual probability of failure for fixed platforms in the Gulf of Mexico and the North Sea is 10^{-3} to 10^{-4}



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7. DESIGN FORMATS

Deterministic Design Format

- In general, we are used to seeing design equations of the form:

Required Resistance = Safety Factor x Nominal Load

or

Allowable Load = Nominal Resistance/Safety Factor

- In this format, all uncertainty is accounted in one safety factor.
  - For e.g., tensile stress shall not exceed 70% of the nominal yield stress of the material.
  - Safety factors are meant to encompass all uncertainty in material, fabrication, load,
  - Factors are usually derived historically, and many times based on judgment and experience
  - Examples include the AISC ASD manual, API RP 2A-WSD, etc ...

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7. DESIGN FORMATS – Contd.

Probabilistic Design Format

- Goal of codes is to produce risk consistent designs -- failure modes with similar inherent risks.
- Numerous ways to incorporate probabilistic analysis into design codes depending on:
  - How much probabilistic information is available to the designer?
  - Are the materials and load environment quantifiable in probabilistic terms?
  - Code to focus on physics of designs, and leave probabilistic aspects transparent to designer?
- Full probabilistic design - one end of spectrum. This method of design can require significant probabilistic analysis on the part of the designer. It is most useful in novel designs.
- Load and Resistance Factor Design (LRFD) format - other end of the spectrum. With this format, the design engineer performs no probabilistic analysis. The design analysis is conducted and the calculated nominal load and resistance values are plugged into a deterministic-looking equation.

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7. DESIGN FORMATS – Contd.

Load and Resistance Design Format

- LRFD is the preferred format by new code writing committees and is of the general form:

$$\phi \times \text{Nominal Resistance} = \gamma_1 \times \text{Nom. Load 1} + \gamma_2 \times \text{Nom. Load 2} + \ldots\ldots$$

  - Resistance factor ϕ represents uncertainty in resistance – typically less than 1.
  - Load factors, γ_1 , γ_2 , ... reflect uncertainty in loading – typically greater than 1.
  - Uncertainty in each load and resistance can be individually accounted for.
  - These load and resistance factors are sometimes termed partial safety factors.
  - Each limit state will have a design equation – partial safety factors will vary for each limit state.

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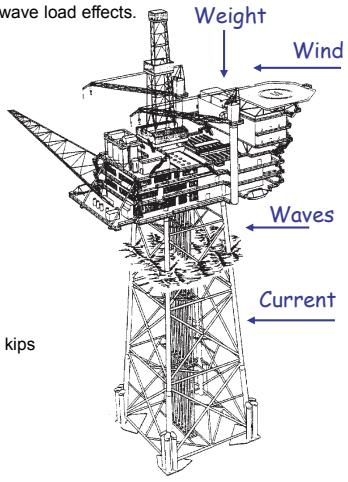
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7. DESIGN FORMATS – Contd.

Example 5

- A tension member on a platform is subjected to dead, wind and wave load effects.
  - Nominal dead load = 10 kips
  - Wave load due to 100 year wave = 50 kips
  - Wind load due to wind associated with 100 year wave = 20 kips
- Nominal yield strength of steel = 50 ksi



Working Stress Design

$f_t \le 0.8\ f_y$

$(10+50+20) / A_{req} \le 0.8\ (50)$

$A_{req} \ge 2.0\ in^2$

Load and Resistance Factor Design

Internal force = Q = 1.1 x Dead + 1.3 x (Wind + Wave) = 102 kips

$f_t \le 0.95\ f_y$

$Q / A_{req} = (102) / A_{req} \le 0.95\ f_y$

$A_{req} \ge 2.15\ in^2$

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8. TYPES OF ERRORS

- **Gross Errors (Blunders)**
  - Missing a mode of failure, different material, numerical error, etc.
  - QA/QC is the only defense against this kind of errors; more important to structural safety than sophisticated analyses; you get what you inspect not what you spec
  - Especially feared in rapid technological developments beyond current experience; suspension bridge failures in the thirties, fatigue ruptures of Comet air planes in the fifties, etc.
  - In early offshore platforms: deck elevation requirements, elevation of first conductor tray below water line, failure of minimum TLP (SeaStar)
- **Systematic Errors (Bias)**
  - Deviations to one side
  - Visual wave records vs. measured significant heights, design formulae based on incomplete theory or experiment, strength properties determined by distorting recording devices, etc.
  - Can hardly be eliminated by inspection
  - Introduced in terms of a mean bias: ratio of actual mean to the nominal calculated value. In design codes the random errors and bias are intended to be covered by the safety factor
- **Random Errors (Design Uncertainty)**
  - Natural variation in load and resistance
  - Can hardly be eliminated by inspection
  - Can be studied quantitatively by means of statistics and the theory of probability

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9. Inspection Intervals

- Aging, corrosion, damage can compromise integrity of facility with time
- Requires periodic attention throughout its operating.
  - Reliability will degrade to a point where probability of joint cracking is of concern
  - If inspection reveals no cracks, perceived reliability of joint is increased immediately
  - The perceived reliability increases but then degrades over time.
  - Probability of detection errors can be introduced and included in the reliability calculation.

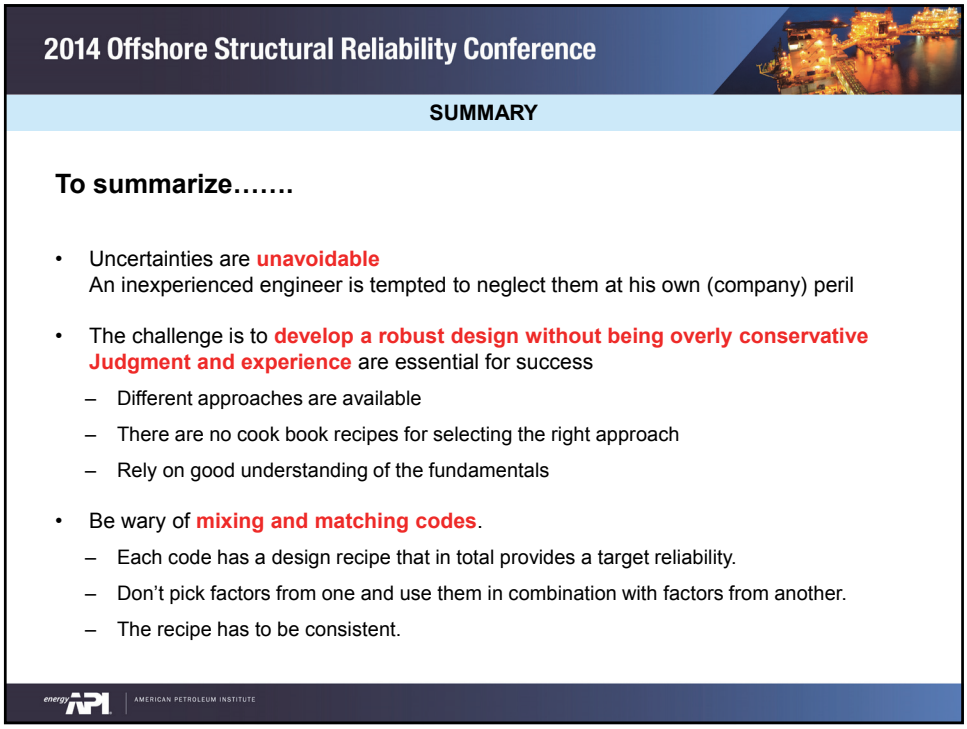


10. INTERPRETATION OF RESULTS

- Reliability results should be judged as notional or relative
- Good dose of experience and engineering judgment.
  - "Garbage in" leads to "garbage out" in reliability calculations.
  - Pay as much attention to input assumptions as to output reliabilities
    - input densities
    - character of the limit state
    - statistics of the random variables
- Results can be plotted in ways that portray wrong conclusions
  Don't rely on plots to make decisions;
  - plot results in other ways;
  - double check the data with hand (or Excel) calculations yourself.

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2014 OSRC
September 16-18, 2014, Houston, Texas, USA

Paper 2.1

HISTORICAL PERSPECTIVE ON THE RELIABILITY DESIGN OF OFFSHORE PLATFORMS FOR EARTHQUAKES

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ABSTRACT

Offshore platforms have been designed to operate in earthquake regions for over 50 years. Early examples include platforms offshore California chiefly designed using equivalent static methods and target reliability similar to those used for onshore structures. The design process was refined in the 1970s and 1980s into a two-tiered approach balancing both strength and ductility as outlined in API RP 2A. The risk-based approach of ISO 19901-2 was introduced in the early 2000s combining the expected level of earthquake ground motions at the platform location and the potential consequences of platform failure. The recently published API RP 2EQ document generally follows the ISO guidance and includes some variations on the risk of manned platforms. This paper summarizes the historical development of earthquake design for offshore platforms focusing on how and why the practice developed, key lessons learned along the way, and some recent and anticipated trends.

EARTHQUAKE HAZARDS

Earthquakes are one of the most devastating natural hazards and annually cause significant causalities and physical damage. Figure 1 shows some of the leading natural disasters since 1980 including economic losses and deaths [The Economist, 2012]. The number of earthquake related deaths exceeds those from other natural hazards – including hurricanes, typhoons, and floods. These deaths are attributed to direct linkage to the shaking earth, tsunamis, landslides, and of course falling buildings and other structures. Yes, buildings and structures. As the saying goes – "earthquakes don't kill people – structures do." However, past earthquakes have shown again and again that properly engineered structures perform well. So as engineers – especially structural engineers – we have to be concerned about earthquakes and we must ensure that our building codes and standards provide structures that are as safe as possible.

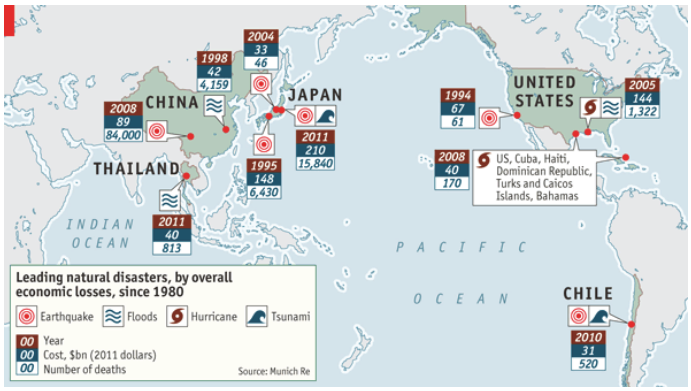


Figure 1 – Leading Natural Disasters Since 1980 [The Economist, 2012]

Fortunately, in terms of offshore structures, there have been no reports of significant damage, injuries or deaths attributed to earthquakes. However, there have been few, if any, "large" earthquakes that have imposed design-level loads on offshore platforms to provide lessons learned to the industry.

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Therefore, seismic design development of offshore platforms has followed the onshore technical approach, with its practical experience of structure performance in real earthquakes, modified as needed with research and development for the unique aspects of offshore structures.

THE EARLY YEARS

The first platform installed in the earthquake prone waters offshore California was platform Hazel in 1956 in 96 feet water depth (Figure 2), followed by platform Hilda in 1960 in 106 feet water depth. Both platforms were installed in California State waters, defined as within 3 miles from shore, and subsequently decommissioned and removed in 1996 [Basavalingandoddi and Mount, 2004]. The first platform installed in Federal waters was platform Hogan in 1967 (still operating). At its peak, the California offshore fleet consisted of 33 platforms. In 2014, there are 27 platforms remaining in operation, ranging in water depths from about 100 feet to 1,200 feet, with four platforms in State waters and 23 in federal waters.

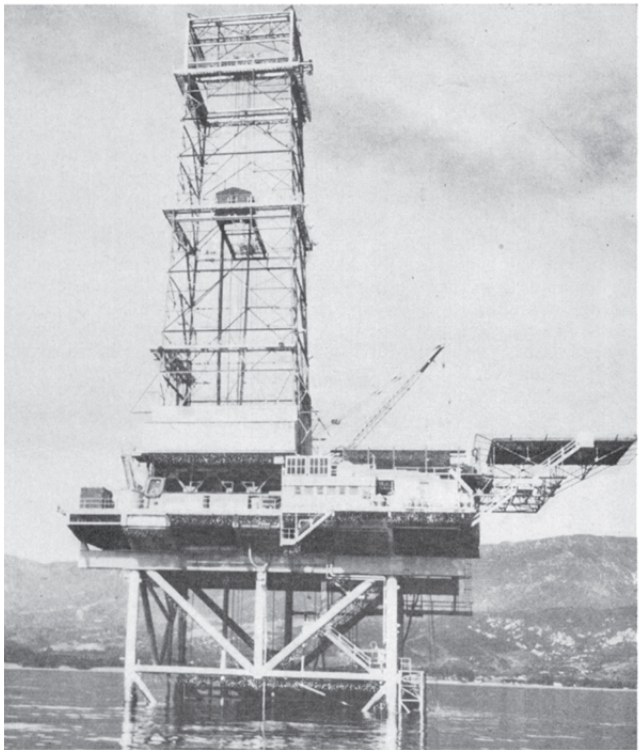


Figure 2 – Platform Hazel – First Platform Installed Offshore Earthquake Prone California 1956

FIRST API GUIDANCE

The first specific seismic design guidance for offshore platforms appeared in the API RP 2A (RP2A) "Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms," 1st Edition, published in October 1969. It contained specific reference to the Uniform Building Code (UBC) including the calculation of an equivalent lateral seismic design load using a special structure design factor for offshore platforms. However, the guidance was limited to only one half page in the sixteen page document. By this time, onshore building designers had also developed some general "good practices" for buildings that were also incorporated into these early platforms, including stronger connections and heavier bracing. They were also predominately 8-leg structures which improves the overall platform redundancy. However, these platforms still followed the general Gulf of Mexico platform design experience and lacked some of the redundant framing (most were k-braced) and deep strong foundations that are prevalent in modern offshore platforms in earthquake regions.

STRENGTH AND DUCTILITY

The RP2A 7th Edition, issued in January 1976, contained a significant update for earthquakes, extending to four pages in the 47 page document, replacing the UBC approach in favor of a two tier design for an "Operating Level" and a "Safety Level." The Operating Level required the platform to be adequately designed for strength to maintain stresses within yield or buckling for the maximum earthquake expected during the life of the structure. The Safety Level required the platform to have sufficient ductility to prevent collapse during a maximum credible event. The annual return periods were not identified for either the Operating or Safety Levels. The Response Spectrum Method was the recommended approach for the Safety Level with maps of the United States provided (from the 1973 UBC) to show the seismic criteria to be used in lieu of site-specific data. The Safety Level design used displacement rather than force as the acceptance criteria. The designer had to demonstrate that the platform would not collapse at a lateral displacement of two times the Operating Level lateral displacement, including P-delta effects, to ensure that the platform had adequate ductility through inelastic deformation. A Time History approach was also mentioned as an option for seismic analysis but was not commonly used due to lack of efficient computer Time History software and hardware. There is also the first mention of "Other Considerations" such as Deck Structure, Rigs, Equipment, Piping, etc., although there is no specific technical guidance.

The RP2A 8th Edition, issued in April 1977, revised the Operating Level and Safety Level terminology to Strength Level and Ductility Level, respectively, although the technical guidance was about the same as the 7th Edition. The industry approach for the two level design requirements was to first "design" for the Strength Level and then "check" that the platform also meets the Ductility Level requirement. In this sense the Ductility Level check was a secondary requirement. The API Seismic Zones and API standard Response Spectra shape found in later editions also made their first appearance in the 8th Edition [Crouse and Stock, 2014].

The RP2A 10th Edition (March 1979) contained significant changes including guidance on computer modeling and some of the original guidance for designing framing, braces, and joints to improve ductility. The Ductility Level "performance goal" was revised, requiring that the structure-foundation system

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should absorb at least four times the amount of energy as was absorbed at the Strength Level and still remain stable. This was typically performed using a pushover-type assessment. For the first time in RP2A there was an extensive ten page commentary section with half the pages devoted to Earthquake Design, including detailed discussion on developing earthquake criteria and conducting analyses. The RP2A 11th Edition, issued in January 1980, contained a much more detailed Earthquake commentary that almost doubled in size to nine pages with an 18 item reference list of notable seismic related papers including Bea [1979], Marshall, Gates and Anagonostopoulos [1977], and Kallaby and Millman [1975].

Renewal of development activity in the Santa Barbara Channel and the lease sales in the Gulf of Alaska, offshore Southern California and the East Coast of the United States in the 1970s initiated a number of earthquake related studies. The most comprehensive of these was the Soil-Pile-Structure Systems (SPSS) [Arnold et. al., 1977]. SPSS, sponsored by 28 oil companies and contractors, assembled the world's experts on the various components of the soil-pile-platform system and concurred on the modeling elements and program specification required to adequately simulate the seismic response of an offshore platform.

LATER RP2A EARTHQUAKE GUIDANCE

The RP2A 15th Edition, issued October 1984, was the last significant change to the RP2A earthquake design approach, and it established the basic seismic design guidance still used today by both API and ISO. Key enhancements included the definition of the Strength Level earthquake as a 200 year return period event and the elimination of the energy absorption approach for the Ductility Level event. Criteria to satisfy this event now included either the use of specific ductility features in the platform framing or an explicit analysis, with the Time History approach recommended. The Ductility Level earthquake return period was not explicitly defined, but was suggested to have a return period of several hundred to several thousand years. The now familiar API figures also first appeared showing jacket vertical framing configurations that provide ductility (X-bracing) and those that do not provide ductility (K-bracing). The overall RP2A document at this time had grown to a total of 115 pages with 20 pages dedicated to earthquakes including 56 technical references.

EXISTING PLATFORMS – THE THIC REPORT

The magnitude 7.1 Loma Prieta earthquake occurred in northern California in 1989, resulting in significant damage to engineered structures – and especially robust freeway structures in San Francisco and Oakland, some 70 miles to the north (Figure 3). This resulted in concerns by the USA offshore regulator, the Minerals Management Service (MMS), that similar damage and destruction could occur to existing platforms located offshore southern California. The MMS, API and industry responded by sanctioning an independent panel in 1991 to prepare a report providing a rational basis for the seismic safety requalification of existing offshore platforms. The panel related recommended performance objectives and requalification methodologies for offshore platforms to those used for onshore structures. The resulting THIC report [Iwan et. al, 1992] was produced in May 1992 and an addendum was published in November 1993 [Iwan, et. al., 1993]. The industry also held several related conferences including the December 1992 International Workshop on Seismic Design and Reassessment of Offshore Structures, held at the California Institute of Technology [Wisch, 1992].





Figure 3 – Damage to San Francisco Region Freeway Structures that Prompted Review of the Seismic Performance of Structures Offshore California [EQE, 1989]

THIC focused on the applicability of onshore seismic design and assessment approaches to offshore structures. It determined that offshore structures requalified using the THIC guidance "should yield platforms whose seismic performance will be comparable to the balance of commercial and industrial buildings onshore". The THIC panel said that offshore structures could be expected to perform "substantially better" than onshore structures because the design, construction and installation are peer reviewed, site-specific design loads are used, and an explicit ductility analysis is performed in addition

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to a design level analysis. Thus it was appropriate to use knowledge of onshore structural reliability to inform the requalification of offshore structures.

The UBC in use at the time [UBC, 1991] specified a design ground motion with a 10% probability of being exceeded in 50 years, which is equivalent to a return period of 475 years (approximately 500 years) and an annual probability of exceedance of 0.002. Additionally, some onshore building codes and practices (at the time in 1993) allowed assessments of existing structures to be performed using 75% of the effective base shear of a new building designed in accordance with UBC. THIC concluded that for a typical hazard curve slope in the California region and the probability level of interest, this 75% base shear value for existing structures increases the annual probability of the hazard by a factor of about 2, giving a 1,000 year return period (i.e., 2x500). Further, Kennedy et al. [1990] and Murray [1991] concluded that because conservative criteria (e.g., load factors, allowable stress, etc.) are used in design, the annual probability of a minimally UBC code-compliant building suffering life-threatening seismic damage during an earthquake is about 0.001 (or 1,000 year return period). Therefore, a 1,000 year return period earthquake was selected as the basis for reassessment of the California platforms as specified in Section 17 of API RP2A [Energo, 2013]. However, this 1,000 year condition for existing structures needs to be revisited by API given the new risk based earthquake engineering approaches in the current API and ISO documents, and the current onshore practice, as discussed below.

ADVANCES IN EARTHQUAKE ENGINEERING FOR ONSHORE STRUCTURES

By the mid-1980s, the RP2A earthquake guidance had received considerable attention from the offshore industry, with nearly 20 percent of RP2A devoted to earthquakes. These procedures represented the *state-of-the-art* for seismic analysis and design of structures at the time – both onshore and offshore. Unfortunately, this was also the end of the era of platforms installed offshore California and further development of the RP2A seismic guidelines dwindled. However, the high profile 1989 Loma Prieta earthquake located near San Francisco, the 1994 Northridge earthquake near Los Angeles and the 1995 Kobe earthquake in Japan initiated considerable research and development in the 1990s for the onshore structures industry, mostly related to improvements in development of seismic criteria, structural analysis and seismic structural design. These approaches were documented in numerous publications by the Federal Emergency Management Association (FEMA) via the National Earthquake Hazard Reduction Program (NEHRP) and American Society of Civil Engineers (ASCE), notably FEMA 356, "Prestandard and Commentary for Seismic Rehabilitation of Buildings", Federal Emergency Management Agency [2000] and related publications. Many of these approaches were eventually incorporated into ASCE 7 [2010] used in much of the USA for building design and internationally, the International Building Code [IBC, 2012] and Eurocode 8 [2012]. See Whitaker and Puskar [2014] for further discussion.

Of particular note in the FEMA work was the further development of "performance based design" concepts with different post-earthquake functionality as shown in Figure 4. While not necessarily a new concept (for example offshore structures already had strength and ductility level performance targets) the onshore industry put forth tremendous development efforts including multiple studies and publications that furthered this line of thinking. With these efforts, the onshore industry established a new state-of-the-art for seismic structural engineering that surpassed the aging RP2A approach.

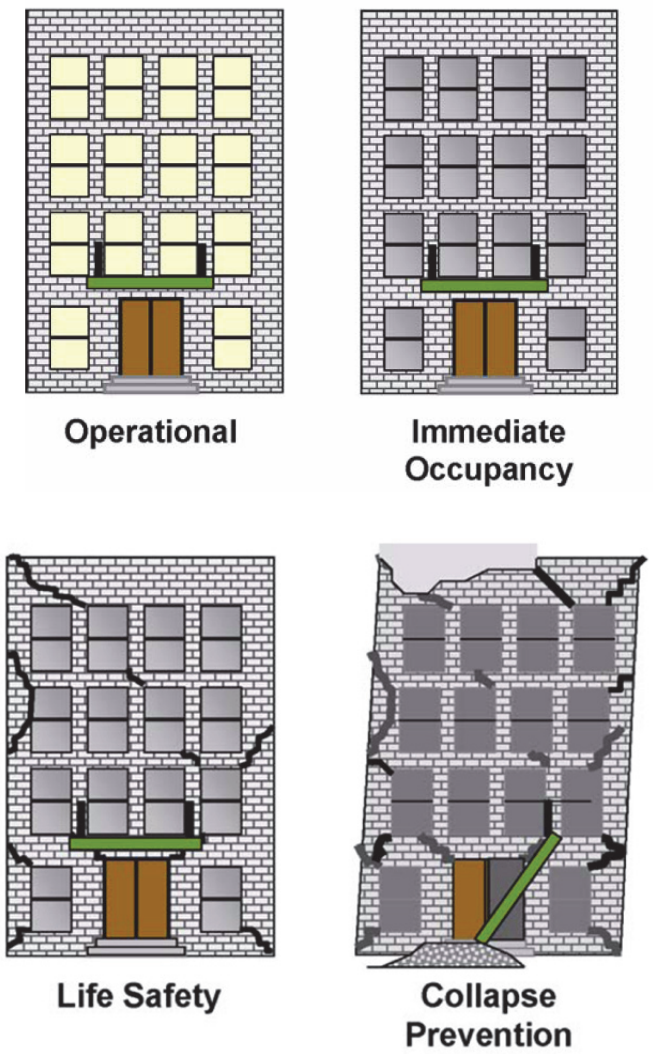


Figure 4 – Performance Based Design Concept [FEMA 389, 2004]

ISO SEISMIC GUIDELINES

In the late 1990s ISO began developing a new standard for earthquake design of offshore structures that eventually evolved into ISO 19901-2, published in 2004 [Banon et. al., 2001]. The ISO guidance borrowed heavily from RP2A and the work by FEMA NEHRP. The two tier design approach was maintained

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although the Strength Level was renamed Extreme Level and the Ductility Level was renamed Abnormal Level to be consistent with the ISO nomenclature for other types of design conditions such as storms. However, the prior API approach of "design for the Strength Level and check for the Ductility Level" was reversed with the emphasis first on establishing a proper ductile platform design. This was accomplished by establishing a target probability of failure for the platform and then, based upon the specific ductility features of the platform, establishing the Strength Level earthquake event. This reversal of approach puts the emphasis on the ductile design of the platform – which is one of the primary findings of the onshore building work in the 1990s – that structures with good ductility design perform the best in earthquakes. ISO added a risk based approach for selecting seismic criteria with High Consequence platforms (manned or unmanned) requiring a lower probability of failure compared to Medium (manned or unmanned) or Low Consequence platforms (see Table 1). ISO also recommended that a Probabilistic Seismic Hazard Assessment (PSHA) be conducted to better define the site-specific earthquake criteria, especially for high consequence platforms.

Table 1 – ISO 19901-2 Probability of Failures According to Platform Exposure Level [ISO 19901-2, 2004]

Exposure Level	Probability of Failure (Pf)
L1 (High)	4.0×10^{-4} (1/2500)
L2 (Medium Consequence)	1.0×10^{-3} (1/1000)
L3 (Low Consequence)	2.5×10^{-3} (1/400)

Other ISO guidelines with earthquake related guidance include ISO 19902 [2007] for fixed steel platforms (containing guidance similar to API RP 2A), ISO 19905 [2012; Hoyle et.al, 2012] for jackups and ISO 19906 [2010] for arctic structures. These documents provide structural guidance specific to the subject structures, and are used in conjunction with ISO 19901-2 to establish the earthquake design criteria.

API RP 2EQ

API RP 2EQ, to be published in 2014, follows the overall guidance of ISO 19901-2 in terms of target reliabilities as well as earthquake structural design features [Younan and Puskar, 2010]. The one significant change is the elimination of the L2 consequence category. As defined in ISO and API, this category allows for platforms that can be manned on an infrequent basis – during daytime hours only for example – but the manning cannot include overnight stays or any types of quartering. When API was considering adoption of ISO 19901-2 for API RP 2EQ, there was concern that ANY platform that is manned – even for short periods – can experience an earthquake when manned and therefore a reduced safety level may pose life-safety risks. After all, earthquakes occur at any time and without warning, meaning there is no time for evacuation. API RP 2EQ therefore only provides guidance for L1 and L3 platforms with the same target reliabilities as in ISO 19901-2 (1/2500 and 1/500 respectively). Recall that the API L3 category platforms are essentially single well caissons only, with little or no production equipment. The only time they should be manned is for very short periods to change batteries for navigation aids or other related minor maintenance. There was some discussion that "small platforms" with limited production can also be considered as "unmanned" – but the provision of any type of production or other equipment on an offshore platform means that there will be periods of manning for repair or update of production equipment and this can amount to days or weeks of time on an annual basis. API determined that, until there is further study, there is no clear definition on what is an acceptable time on an annual basis for personnel to be on the platform for maintenance or other activities to allow the platform to be considered "unmanned" from the standpoint of earthquake design criteria. This change is also under consideration for the pending update to ISO 19901-2. API's long term plans call for the target reliabilities of various types of offshore facilities to be consistent for key risks such as storms and earthquake and will account for manned and unmanned situations. This will be documented in a new document called API RP2 GEN. Further details are discussed in Wisch and Verret [2014].

ISOLATION SYSTEMS

Seismic isolation systems serve to reduce earthquake effects acting on a structure. Although not widely used for offshore structures, there are several examples where such devices have been used in order to control earthquake effects. These include the Chirag 1 platform in the Caspian Sea, the Sakhalin I Gravity Base Structure (GBS) Arkutun-Dagi [Bruen et. al., 2009], the two Sakhalin II GBS, Lunskoye A and Piltun-Astokhskoye B platforms [Clarke et. al., 2005]. The Sakhalin platforms use a friction pendulum bearing (FPB) system, to reduce earthquake actions at the deck level with an FPB located at each of the four deck connection locations (Figure 5). ISO 19906 for arctic structures contains guidance for the use of isolation systems on offshore platforms. The guidance is in an arctic document since the predominant use thus far for such systems has been on massive concrete GBS which are well suited for arctic regions to resist large ice loads. A GBS tends to transmit seismic motions to the deck level via lateral and vertical motions as well as rocking of the concrete base structure, making it a good candidate for an isolation approach. Note that, even with these isolation systems, the target reliabilities for these structures are the same as for structures without these systems. The isolation systems are an alternative means of controlling earthquake actions versus structure response alone.

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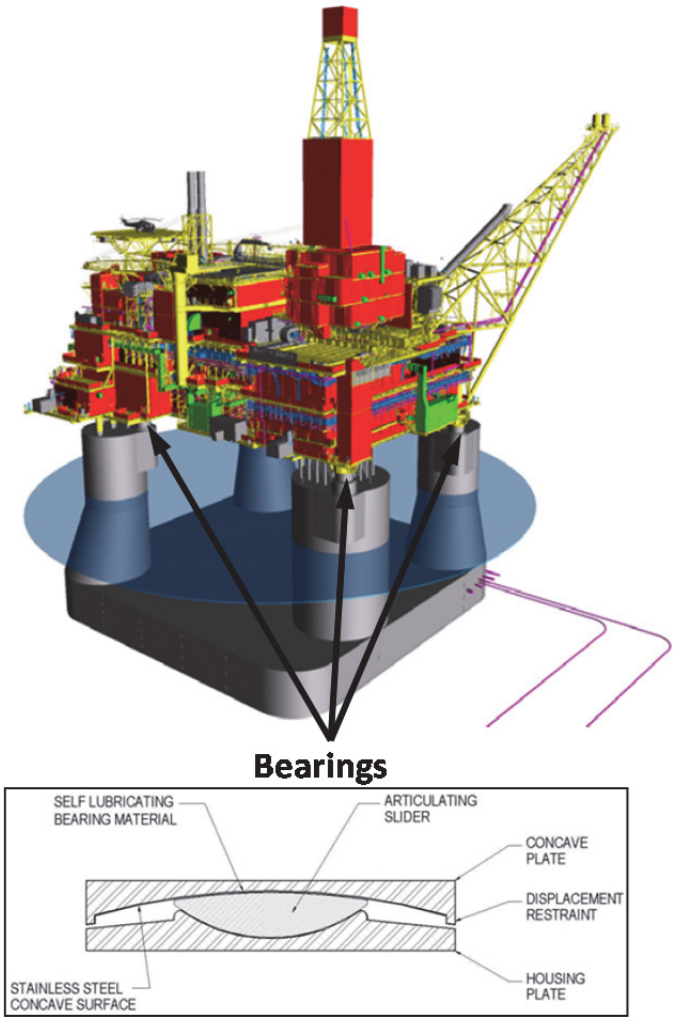


Figure 5 – Friction Pendulum Bearing Seismic Isolation Used on Sakhalin GBS [Clarke et. al, 2005]

MANAGING EARTHQUAKE RISK

Earthquakes occur without advanced warning putting personnel at risk. Several platform owners have been managing earthquake risks by developing post-earthquake action plans based on structural studies of the amount of damage that can be caused by different levels of earthquake shaking. These plans are used in conjunction with information on the specific earthquake once it occurs in order to make informed decisions as part of the post-earthquake response plan.

Not all earthquakes that shake a platform will shut-in operations or result in damage that requires repair, while others may be large enough to lead to significant damage both above and below water. Personnel onboard are not in a position to distinguish between events that have life safety consequences and those that are not unless that platform has been damaged to such an extent that evacuation is the obvious choice. The action chart provides a tool with which personnel can readily distinguish between minor and significant events and respond appropriately.

Damage may have occurred underwater where it is not apparent from the topsides. Large earthquakes that are likely to damage a platform will have significant aftershocks, usually within a few hours to a few days. If there is damage below water, then this damage combined with the aftershock may topple the platform. There will be insufficient time to mobilize an underwater inspection to survey for damage, and if damage is found, insufficient time to take appropriate action. If the platform is manned then personnel may be at risk. Platform owners also do not want to evacuate personnel (which in itself has risks) or mobilize inspection equipment for smaller earthquakes that likely caused no damage.

Figure 6 shows a generic form of a post-earthquake action curve with three possible post-earthquake actions. The first is *No Action* for smaller earthquakes (measured as a combination of magnitude of earthquake and distance from the platform) that may shake the platform with no resulting damage. The second is to *Inspect* the platform since this level of moderate earthquake may have caused damage but is unlikely to lead to collapse given an aftershock event. The third is to *Evacuate* for the most severe earthquakes that likely caused damage, especially below water, and it is prudent to evacuate personnel prior to any aftershock. The development of such an action curve is complex and is based on a combination of the magnitudes of earthquakes that can occur at different distances from the platform, along with the structural performance of the platform at different levels of shaking available from the Strength and Ductility Level analyses. Further details are provided in Puskar, et.al. [2013] and Dolan, et. al. [1992]

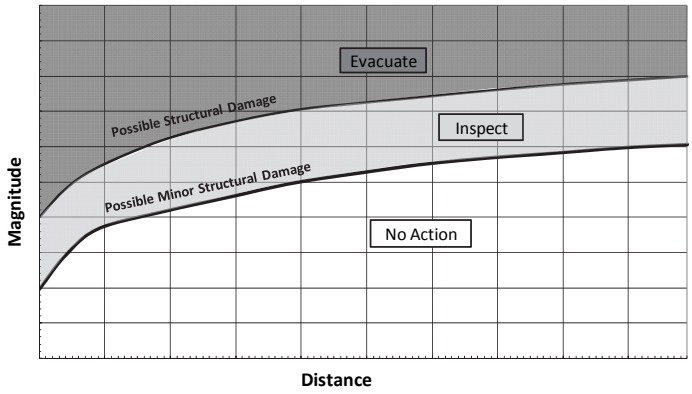


Figure 6 – Example Post-Earthquake "Action" Chart [Puskar, et. al., 2013]

SUMMARY

Between API and ISO there are now over 50 years of seismic design guidance and associated experience with scores of platforms analyzed and installed using these approaches. API and ISO now provide multiple guidelines and recommended practices for designing various types of platforms for earthquakes. These documents build upon the experience from offshore operations as well as the experience of onshore

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