Developments in extreme metocean loads and assessment of fixed offshore structures.

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ABSTRACT

The LOADS JIP [1], performed by Offshore Consulting Group and Imperial College from 2015 to 2023, and TotalEnergies' AWARE project [2,3], (from 2013 to present), can be considered as providing an improvement and extension of the following methods, resulting in more realistic time-histories of the magnitude and duration of load during an extreme wave event:

a) TVM (Tromans and Vanderschuren Method, 1995 [4]) for determining extreme metocean loading on spaceframe jacket structures (including jack-up legs) and:
b) API silhouette method (based on tank tests performed by Finnigan and Petrauskas in 1989 and reported in 1994 and 1997 papers [5,6]) for determining wave loading from wave crests that inundate the deck or topsides.

NOTE: "extreme" is used here to mean the statistical theory of extremes and not, as in some codes of practice, the magnitude of load having an annual probability of exceedance of 1/100. Extreme wave events can occur in various sea states and include - a) a wave event (with a moderate probability of exceedance, say 1/100) in a sea state with small annual probability of exceedance, say 1/1000 and, b) a wave event (with a small probability of exceedance, say 1/1000) in a sea state with moderate probability of exceedance, say 1/100.

ISO 2394:2015 formalises the approaches and methods that can be used for limit state verification. Typically, the Partial Factor Method (PFM), which is method belonging to the Semi-Probabilistic approach (S-PA), is used for design. However, methods belonging to the Reliability-Based approach (R-BA) or Risk-Informed approach (R-IA) can be good options for assessment of existing structures if the PFM fails to demonstrate that acceptance criteria have been met, as they are more "structure specific" and thus less conservative than the PFM, even acknowledging they are more difficult and time consuming to apply.

KEY WORDS: Extreme metocean loading, Approaches for limit state verification.

1 - INTRODUCTION

It is not surprising that such improvement and extension has been achieved, because over the 30 years since the TVM and silhouette method were published, the following developments have occurred:

a) computer power is orders of magnitude greater than in 1995.
b) techniques for implementing statistics of extremes have been published (e.g., extrapolation method of Heffernan and Tawn [7,8]; application of Bayesian inference by Hamiltonian Monte Carlo [9]).
c) measurement and tank testing techniques for extreme metocean conditions are more accurate, and:
d) CFD has developed and can now model extreme wave events.

The TVM and silhouette method can provide similar wave loading as the LOADS/AWARE methods for platforms with certain configurations that are exposed to certain environmental conditions, however, the LOADS/AWARE methods are more generally applicable and can result in more (or less) onerous loading for specific platform configurations that are exposed to specific environmental conditions.

The purpose of this paper is to describe:

a) the latest developments in determination of extreme metocean loading on fixed offshore structures and
b) the latest developments in methods for limit state verification, where limit state verification is explicit or implicit demonstration that the risks (life-safety, environmental-pollution & business-disruption) are tolerable and ALARP.

2 - BASIS OF WAVE-IN-DECK LOAD

The time history of load imparted to a deck or topsides is due to the rate of change of momentum of the volume of water in the inundating part of the wave crest. The rate of change of momentum depends on the porosity (or blockage) of the deck while the momentum of the inundating volume of water depends on:

a) crest height relative to the bottom of steel elevation (i.e., the inundation)
b) crest steepness and degree of wave breaking (i.e., non-breaking, spilling or plunging).
c) crest width due to degree of wave spreading (i.e., degree of short-crestedness)

Two wave inundation events with the same crest height will typically result in very different magnitudes of wave-in-deck peak load as shown in Figure 1 where the arrows are water particle velocity direction, the colour indicates the water particle speed, and the grey box indicates the inundated part of the deck. The plot on the left is a non-breaking wave event with small steepness resulting in a larger volume of water inundating the deck compared to the right plot which represents a steep breaking wave event.

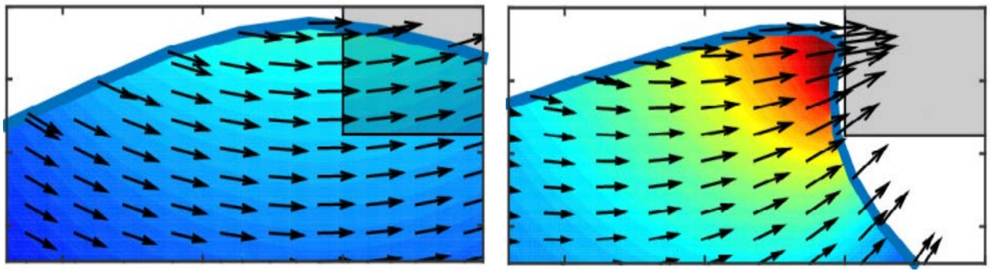

Figure 1 – plots from [1] illustrating parameters affecting wave-in-deck load.

Wave-in-deck load is therefore a stochastic process that can only be properly assessed using long-term and short-term statistics based on data sets that are sufficiently large allow a distribution to be fit to events with annual probability of exceedance in the range of 10^{-4} to 10^{-5}. Typically, tank tests are performed at a 1/100 scale, however, the NSTC JIP (Near Shore Test Centre) plans to perform a field test at a scale of 1/10.

3 - BASIS OF SILHOUETTE METHOD (FOR WAVE-IN-DECK LOAD)

The silhouette method is based on tank tests performed by Chevron in 1989 at Canada's National Research Council in Ottawa. [6] provides a detailed description of the model and tests and states:
"Multiple wave time histories 3 hours in length (prototype scale) were numerically simulated from a typical Gulf of Mexico design wave spectrum. These time histories were screened to identify wave segments 3 minutes in length (prototype scale) with the following criteria: a) the segments contain 2 to 3 waves with a crest height between the bottom of the scaffold and top of the main deck; and b) at least 2 to 3 waves separated these large crests for statistical independence. **Only those wave segments which satisfied the above criteria <u>and were not breaking</u> were used in the model tests.**" The results presented in [6] show a significant variability in peak wave-in-deck load but a relatively small bias (Figure 2 left plot) and typically, the peak wave-in-deck load calculated by the API silhouette method is conservative compared to test data (Figure 2 right plot).

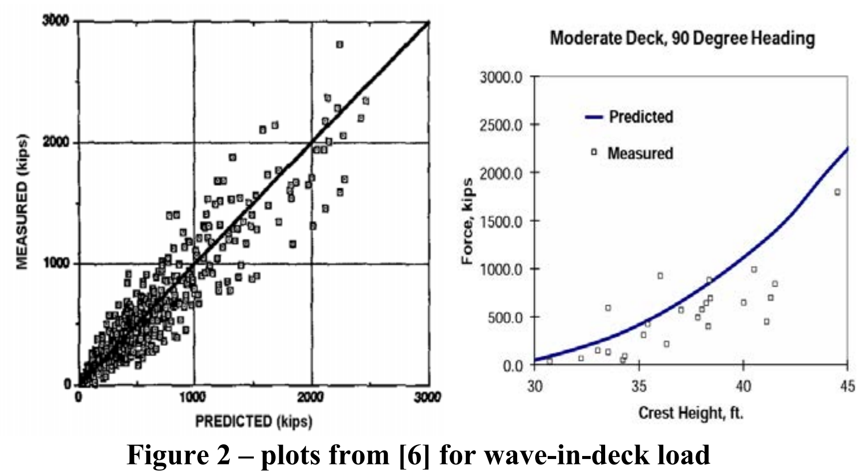

Figure 2 – plots from [6] for wave-in-deck load

The long-duration tank tests performed in the AWARE project and LOADS JIP have revealed that breaking wave events (both spilling and plunging) occur routinely in extreme sea states, even in deep water. Thus, by ignoring breaking waves, the 1989 tests are missing critical data that will increase the variability of the wave-in-deck load as a function of crest height inundation compared to that shown in Figure 2.

4 - BASIS OF TVM (FOR WAVE-IN-JACKET LOAD)

The annual probability of the load imparted to a jacket (or to jack-up legs) exceeding a given value (i.e., the hazard curve) is determined by the convolution of the:

a) long-term distribution of the sea state (where the sea state is typically characterised by probability of exceedance of storm peak Hs).
b) short-term distribution of the wave events in a given sea state.

The TVM has an analytical basis (rather than a numerical sampling basis) that is illustrated by the steps in Figure 3 that result in the hazard curve.

$$\text{hazard curve} = v \times \int_{storms} (1 - \text{short term distribution} \mid \text{long term density}) \times \text{long term density} \times dl_{mp}$$

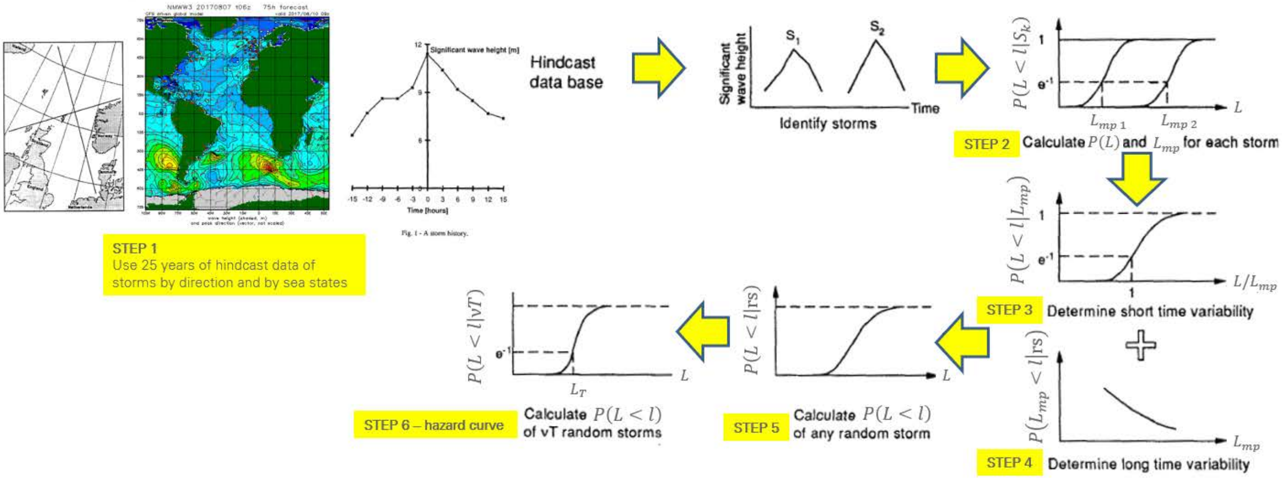

Figure 3 – illustration of the steps in the TVM, from [4], to determine the hazard curve for wave-in-jacket load (*L*).

Step 5 of the TVM, shown in Figure 4, determines the load from the hindcast data (that has a relatively short duration, typically 25 to 50 years) and then extrapolates the long-term distribution of load to the return period required (typically 10k years to 100k years).

$$P(L > l|\alpha, \text{storm}) = \int_{L_{mp\,min}}^{L_{mp\,max}} \left[1 - P\left(L \le l \middle| L_{mp}\right)\right] \times p\left(l_{mp}|\alpha, \text{storm}\right) dl_{mp}$$

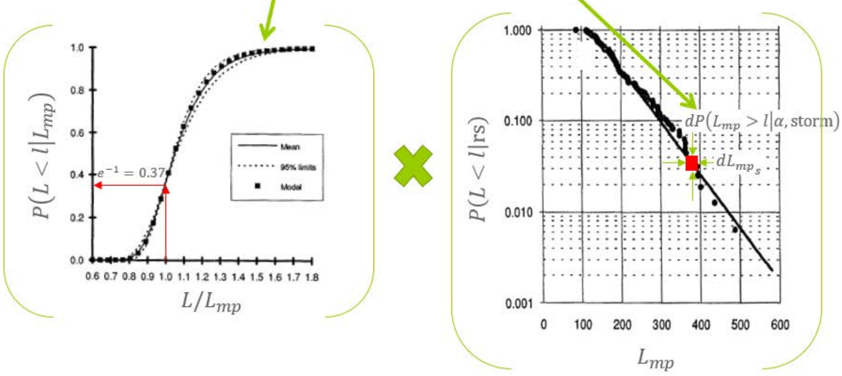

Figure 4 – illustration of step 5 in the TVM and the extrapolation of probability of exceedance of load to longer return periods than the hindcast data.

The TVM was originally based on use of deterministic NEWWAVE theory, however, further work by Shell showed that (deterministic) Stokes waves produced very similar results.

5 - BASIS OF LOADS JIP (FOR WAVE-IN-DECK LOAD)

The LOADS JIP (and the AWARE project) performed tank tests for many different sea states ($Hs, Tp, \gamma, \sigma_\theta$) and water depths, with each sea state being simulated for a very long duration, to determine the probability of exceedance of crest height. Figure 5 shows an example with and without breaking for directionally spread (σ_θ= 15°) sea states defined by a JONSWAP spectrum with γ =2.5, Tp =16s, d=125m and Hs = 17.5 m. Note that the test data (red and black dots) extend to a probability of exceedance of 10^{-4}.

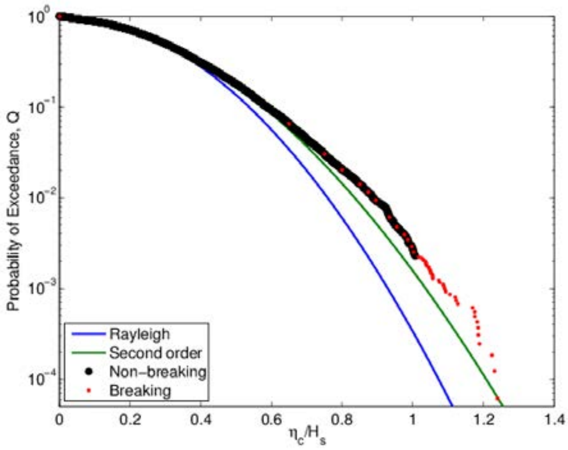

Figure 5 – example, from [1], of data generated in the LOADS JIP for annual probability of exceedance of wave crest η_C (normalised by Hs).

Due to the large amount of testing in both LOADS JIP and AWARE, the resulting crest statistics and wave-in-deck loads are based on the full aleatory variability in the crest shape and the crest kinematics, as illustrated in Figure 6, whereas, by discarding breaking waves, the 1989 Ottawa test results are not representative for spilling or plunging wave events.

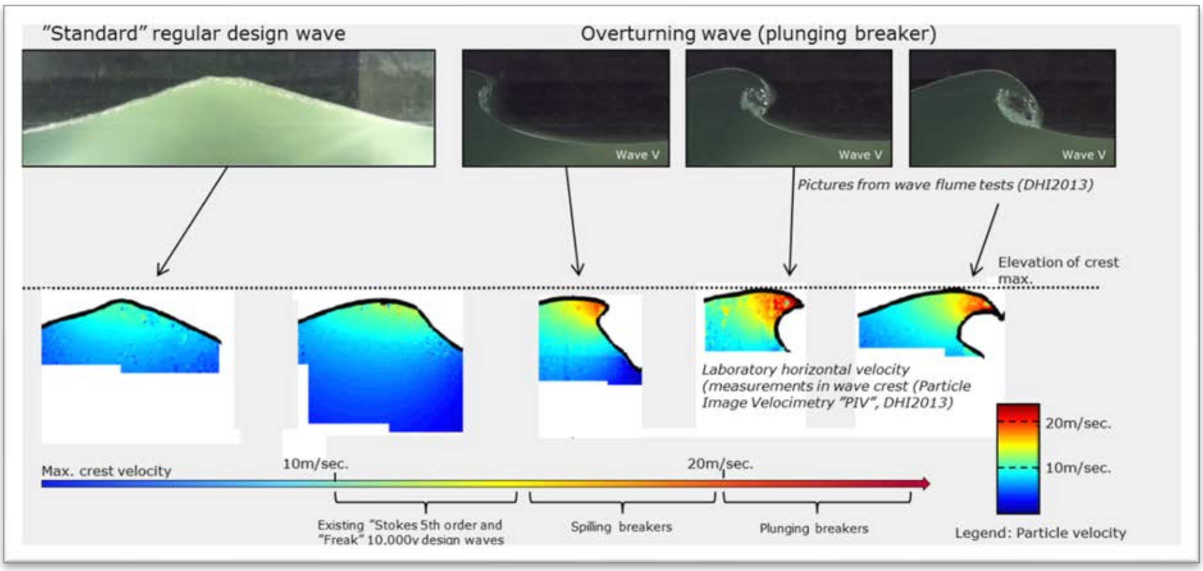

Figure 6 – example, from [2], of data generated in the AWARE project (similar for LOADS JIP) for crest height and crest kinematics

Having measured the crest heights and crest kinematics, the LOADS tests were repeated on an identical basis but with a scale model of the deck suspended on a rigid loadcell above the tank to determine the peak wave-in-deck load and the time history of the load. Figure 7 shows typical time history plots for wave-in-deck load.

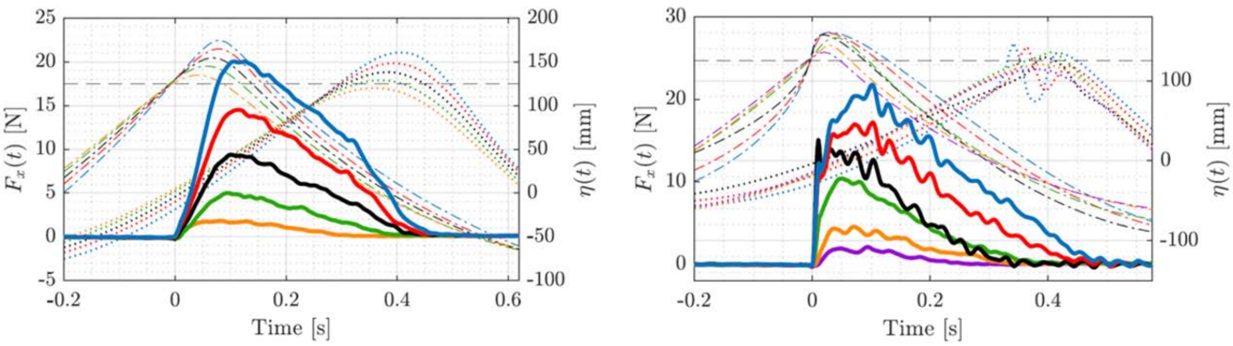

Figure 7 – example, from [1], of time history of wave-in-deck load from tank tests (10mm test scale =1m real scale & 1 sec real scale = 0.1 sec test scale)

Horizontal WID loads, $F_x(t)$, for increasing levels of inundation, $\Delta\eta$; the dashed-dot lines defining $\eta(t)$ on the front-face of the topside structure and dotted lines $\eta(t)$ on the back-face. (a) *JONSWAP* spectra (T_p=1.6s, γ=2.5, σ_θ=0°) in d=1.25m with $\Delta\eta$=10mm, 20mm, 30mm, 40mm and 50mm (at laboratory-scale, all non-breaking), (b) *JONSWAP* spectra (T_p=1.4s, γ=2.5, σ_θ=0°) in d=1.25m with $\Delta\eta$=10mm (non-breaking), 20mm (limiting), 30mm (spilling), 30mm (over-turning), 37mm (over-turning), and 40mm (over-turning)

Figure 8 shows, the distribution of horizontal load, F_x^{WID}, expressed in terms of the elevation of an incident wave crest measured at the centre of jacket, $\eta_{crest,centre}$. The fact that non-zero values of F_x^{WID} occur for $\eta_{crest,centre} < h_{deck}$ reflecting that $\eta_{crest,centre} \neq \max_{deck\ area}(\eta_{crest})$

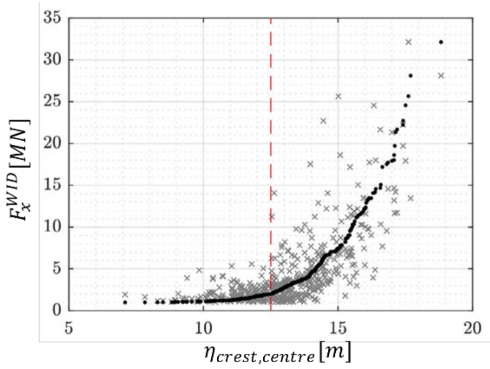

Figure 8 – example, from [12], $F_x^{WID}(\eta_{crest,centre})$ for a 60m×60m topside structure with an average porosity of 63%.

[×] wave events plotted against their corresponding loading magnitudes,

[•] a quantile–quantile (Q-Q) ranked plot defining the most probable load for any $\eta_{crest,centre}$

[---] h_{deck} deck bottom of steel elevation above still water level.

Figure 9 shows the percentage of cases where Silhouette method (for C_d =2.5 but [1] contains similar plots for other values of deck porosity/ congestion) under-predicts or over-predicts the wave-in-deck load compared to LOADS JIP tank test data based upon deterministic focused wave groups for a topside structure with Porosity =0%; Plots (a), (b) compare API silhouette method with LOADS JIP, while plots (c), (d) compare the Santala modified silhouette method [10] with LOADS JIP. It is clear from Figure 9 that, both the API and Santala modified silhouette method, tend to under-predict the wave-in-deck load, i.e., the blue histogram bars exceed 50% of the compared cases for every wave steepness group. However, this is not always the case, i.e., the red histogram bars show the silhouette method can over-predict the wave-in-deck load for 8% to 40% of the compared cases, depending on the wave steepness. This means the silhouette method cannot be used as a simple and conservative method of determining the wave-in-deck load because the confidence in result being conservative can be as low as 8%. In random seas the aleatoric variability of the waves results in the area effect and also different degrees of wave breaking. Breaking waves (spilling or plunging) result in larger loads than those produced by a focussed wave (and breaking waves were excluded from the test data that forms the basis of the Silhouette method).

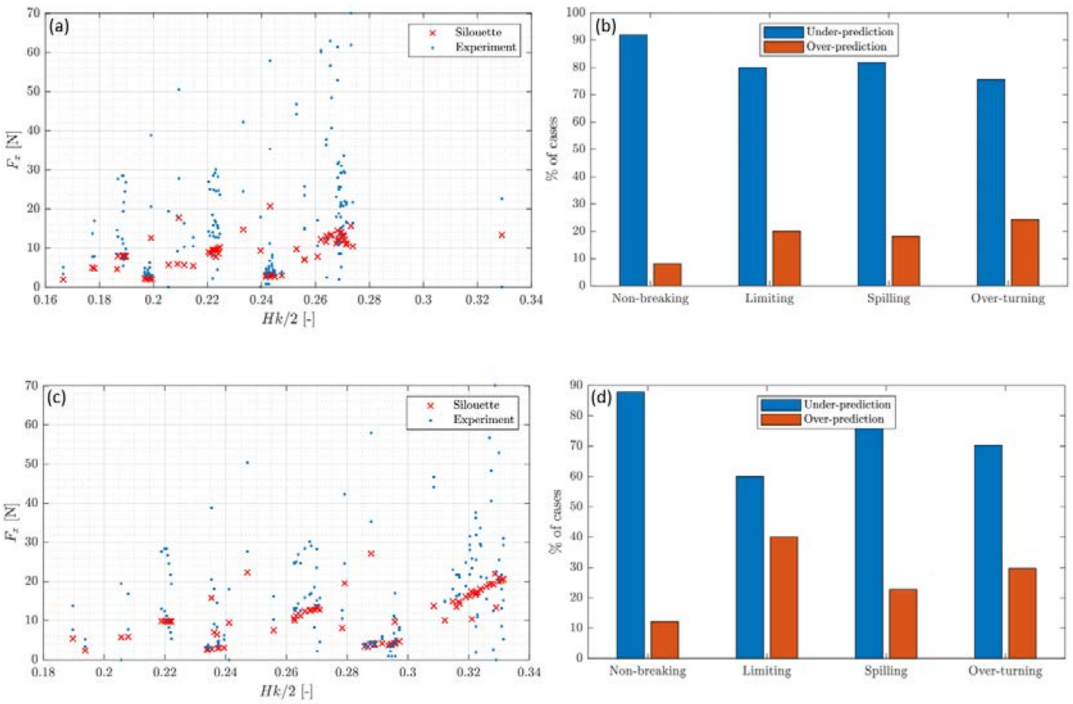

Figure 9 – from [1], showing under/over prediction of Silhouette method (with C_d =2.5) compared to laboratory data based upon deterministic focused wave groups for a topside structure with Porosity =0%; (a), (b) calculated using API silhouette method and (c), (d) calculated using the Santala (2017) modified silhouette method

Figure 10 shows an example of a platform where the annual probability of topsides WID load is less than 4×10^{-3}. The WID load shown by the red circles has been calculated using the silhouette method with a wave crest elevation based on the LOADS crest distribution for the given probability of exceedance. For this case, the Silhouette method is approx. 75% of the LOADS WID load.

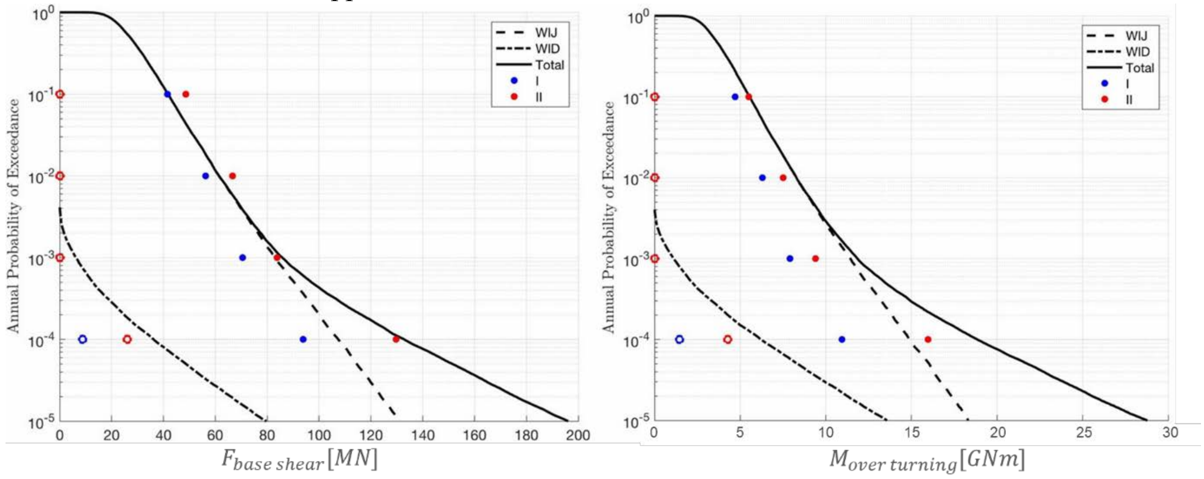

Figure 10 – from [11], where the solid lines are the LOADS JIP hazard curves for base shear (BS) and overturning (OTM), with the WID component shown by the dash-dot line. The red circles are WID BS & OTM calculated using the silhouette method.

6 - BASIS OF LOADS JIP (FOR WAVE-IN-JACKET LOAD)

The LOADS JIP method and the TVM both produce hazard curves, an example of which is shown in Figure 11. The x-axis of a hazard curve is the intensity measure (IM), that is a scalar representation of the magnitude of a hazardous event (i.e., an extreme wave event). The IM in Figure 11 is base shear, normalised by the base shear that is exceeded with a return period of 100 years (E_{100}). It can be seen that the platform has an annual probability of exceedance of zero wave-in-deck load of 6×10^{-4} and, the base shear that is exceeded with a return period of 10k years (E_{10k}) is $1.9 \times E_{100}$. Other IMs can be used for the hazard curve, the most ideal IM is the one where the fragility curve (see Figure 14) has minimum dispersion (i.e., maximum steepness).

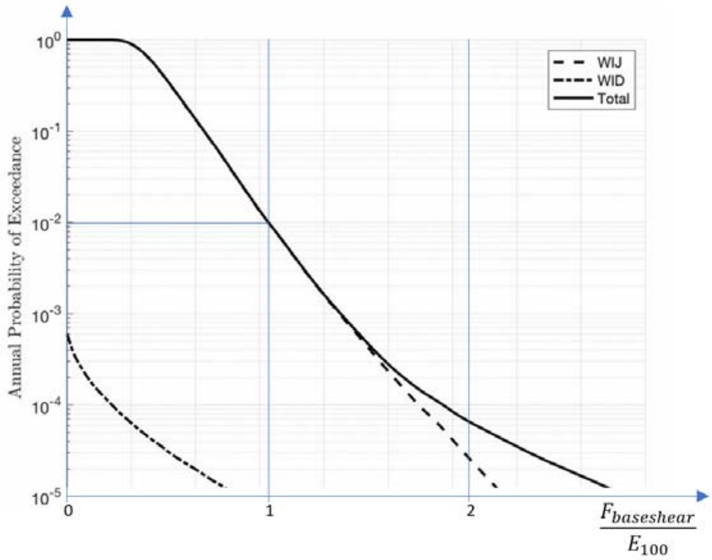

Figure 11 – example, from [11], of hazard curve for base shear

Figure 12 illustrates one of the key differences between the LOADS JIP method and the TVM. The start point for both methods is the upper left, with input data from metocean hindcasts. Hindcast data is typically produced from data gathered from 25 years to 50 years of metocean conditions, and therefore, represents the "body" of the probability of exceedance rather than the upper tail that contains the extreme events with annual probabilities in the order of 10^{-4} to 10^{-5}. The TVM converts the metocean data into a single variable (base shear) and then extrapolates to the extreme values using a shifted exponential distribution while also (analytically) convolving the short-term and log-term distributions of base shear. The LOADS JIP determines the hazard curve by firstly extrapolating to the extreme tail of the joint probability distribution function (j-pdf) of the metocean variables. This step uses the Heffernan and Tawn [7,8] extrapolation methods that were developed 9 years after publication of the TVM. The probability of exceedance of the base shear is determined by sampling the metocean variables from the tail of the j-pdf, that are used to create samples of time histories of extreme wave events, that are used to perform time history load analysis of the jacket and deck to determine the peak base shear for each wave event. The (stratified Monte Carlo) sampling is performed tens of thousands of times and therefore the hazard curve can be readily determined by order statistics.

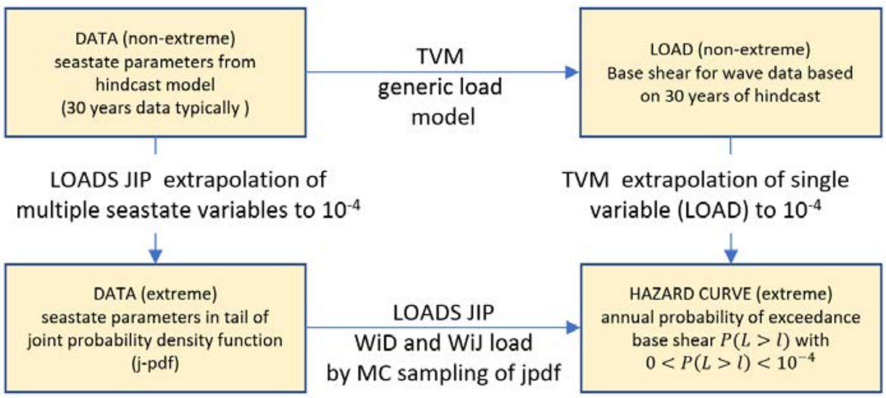

Figure 12 – difference in TVM and LOADS JIP

The modern statistical and sampling methods (exploiting the computer power now available in a high spec laptop) used in the LOADS method mean that the variability in wave-in-deck load due to variability in steepness, crest shape and crest kinematics in of the wave events is directly included.

In addition, the variability in the wave-in-jacket load (primarily due to the large variability in kinematics above still water level) is also directly included in the LOADS method. Figure 13 illustrates the variability in the shear force as a function of jacket elevation (using a proxy of velocity squared for drag shear force) due to 100 sampled wave events, conditioned on the total base shear being equal for all wave events.

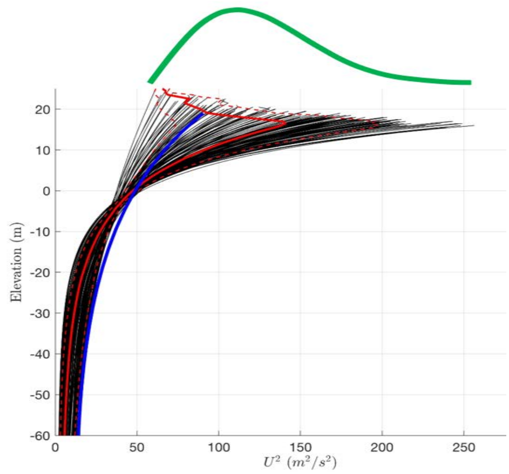

Figure 13 – example, from [1], of square of horizontal velocity (proxy for drag load) versus depth for waves of different steepness, all waves produce the same base shear)

The blue line in Figure 13 shows the shear force as a function of elevation produced by a wave Stokes wave. Therefore, the variability in shear force at a given elevation, for a given value of base shear, due to the stochastic nature of wave events cannot be represented by regular wave theory. This (aleatory) variability in shear force can result in a larger annual probability of jacket collapse due to a larger contribution to collapse probability from failure modes near the SWL or collapse modes lower in the jacket governed by overturning.

A sub-set of the samples of time histories of wave-in-deck load and wave-in-jacket load that were used to create the hazard curve are used to perform nonlinear time history response analysis of the platform structural model. Figure 14 shows a fragility curve, which is a statistical distribution fit to the number of platform collapses and non-collapses conditional on base shear (black dots on Figure 14). The fragility curve represents the probability of platform collapse conditional on the wave event occurring having a given base shear. The blue shading is the uncertainty in the statistical fit, which is included in the final probability calculation. The integral of HdF gives the expected value of the annual probability of jacket collapse.

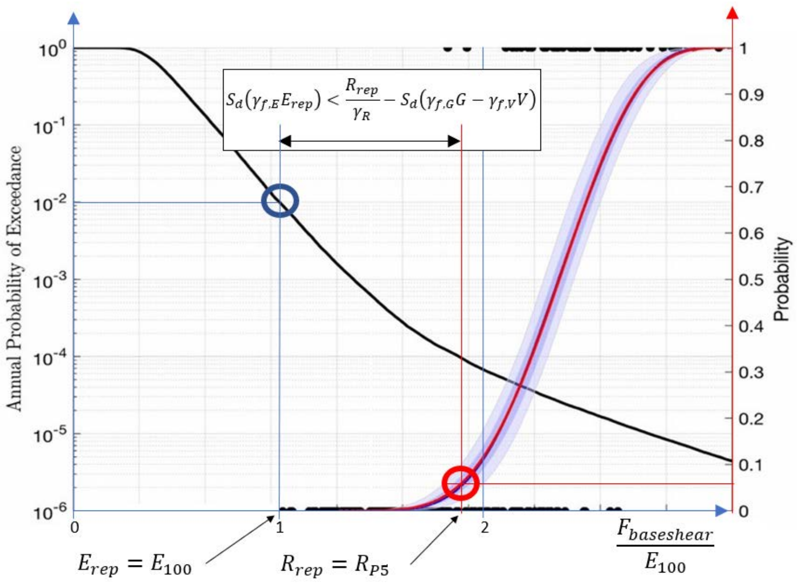

Figure 14 – example, from [11], of hazard curve and fragility curve for base shear

Specific extensions and improvements provided by the LOADS JIP are:

1) more transparent and consistent inclusion of epistemic uncertainty in the extrapolation of extreme storm peak sea states (by use of Bayesian inference with priors based on limiting values of physical quantities rather than on distribution parameters)
2) more accurate determination of extreme wave events by sampling wave events from the extremes of the joint probability density function of sea state variables.
3) more accurate determination of wave-in-jacket loading above still water by stochastically accounting

for the non-linear kinematics in the wave crest.

4) more accurate determination of wave-in-deck loading by stochastically accounting for the momentum of fluid inundating the deck. i.e., the volume of fluid in the wave crest inundating the deck and the velocity of that fluid is determined by the wave steepness, the wave shape, the degree of breaking (spilling or plunging) the width of the wave crest and the non-linear kinematics in the wave crest.
5) more accurate determination of the metocean hazard curve for the structure by aggregation of the above by variance reduction Monte Carlo sampling (stratified sampling and importance sampling).
6) more accurate determination of individual hazardous wave events by deaggregation of the hazard curve over the critical range of the intensity measure.
7) determination of the structure's fragility curve by fit of a distribution to results from nonlinear time history analysis of the structure's response to the above individual hazardous wave events.

The difference in the calculated probability of collapse of a structure from use of the TVM compared to the LOADS method is currently being determined by application of the LOADS method to a variety of fixed jacket structures in the Energy Institute study S2027 [11]. For some structures and environments little difference is expected, while for other cases more significant differences have been found. A sufficient number of examples are planned to be completed by 2024 thus allowing guidance on which type and form of structure and in which environments the LOADS method will result in the largest differences from the TVM.

7 - APPROACHES & METHODS FOR LIMIT STATE VERIFICATION

Approaches for limit state verification were generalised and formalised in the 2015 issue of ISO 2394. Figure 15, presented by Marc Maes at the 2016 OSRC, illustrates the approaches and their relationship.

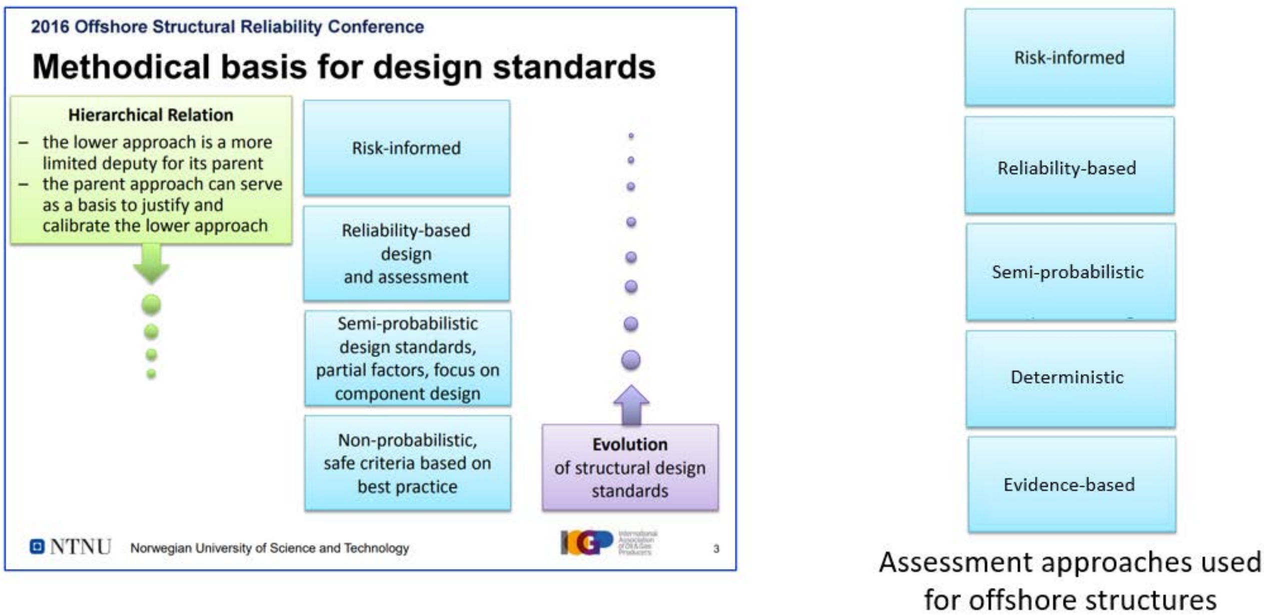

Figure 15 – illustration of ISO 2394 approaches for limit state verification (and their relationship)

All decisions on the fitness-for-service of a structural design or assessment are, in reality, risk-informed (i.e., RIDM – Risk-Informed Decision Making), as explained in the following:

- limit state verification by a risk-informed approach **explicitly calculates the risk** magnitude for each risk type. i.e., **explicitly** calculates the expected value of the annual probability of exceeding the relevant limit state and **explicitly** calculates the expected impact (magnitude of loss for each consequence type) if the structure exceeds the limit state.
- limit state verification by a reliability-based approach **explicitly** calculates the expected value of the annual probability of exceeding the relevant limit state but **implicitly** uses a conservative value of the expected impact (magnitude of loss for each consequence type) if the structure exceeds the limit state.

Therefore, it **implicitly calculates the risk** magnitude for each risk type.

- limit state verification by a semi-probabilistic approach **explicitly uses calibrated partial factors and event probabilities.** However, due to the calibration, this approach is **implicitly related to the risk** magnitude for each risk type. i.e., this approach **implicitly** uses the expected value of the annual probability of exceeding the relevant limit state and **implicitly** uses a conservative value of the expected impact (magnitude of loss for each consequence type) if the structure exceeds the limit state.

The table in Appendix A summarises the approaches, methods, options, and performance criteria for limit state verification.

The steps involved in the semi-probabilistic, reliability-based, and risk-informed approaches are shown in Figure 16. The outcome is Risk-Informed Decision Making (RIDM) either explicitly, or implicitly, or by a combination of implicit and explicit.

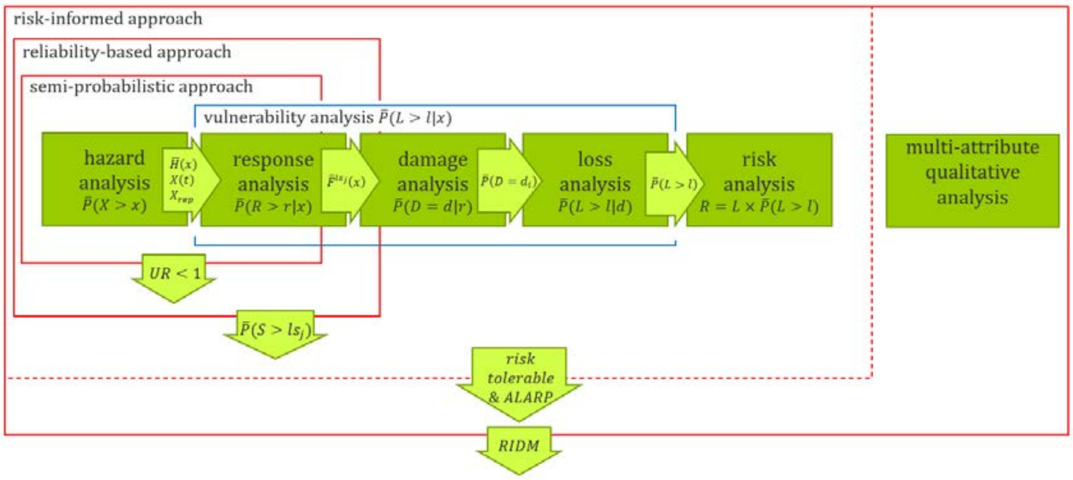

Figure 16 – illustration of performance-based design (showing relationship between approaches)

8 – EXAMPLE: ASSESSMENT OF A FIXED PLATFORM BY EIM (USING LOADS JIP)

Verification of the collapse limit state of the platform shown in Figure 17 has recently been performed in an Energy Institute study [11] using the EIM (Exact Integration Method), which is categorised as a reliability-based approach – see Appendix A. The hazard curve and fragility curve, as shown in Figure 14, were determined using the LOADS JIP method. The HdF integral included epistemic uncertainty in the hazard curve (from statistical extrapolation) and epistemic uncertainty in the fragility curve (from uncertainty in yield stress and uncertainty in the ISO code capacity of components arising from variability in test results of component collapse). The calculated expected value of the annual probability of jacket collapse was 8×10^{-5}.

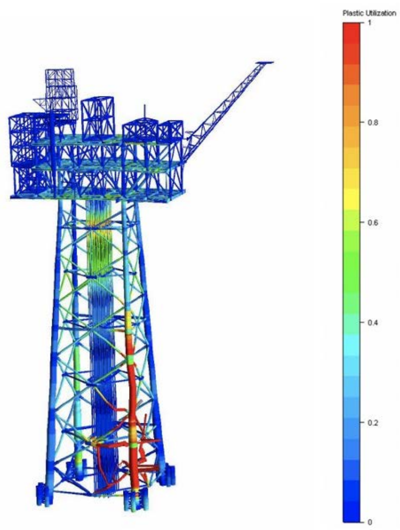

Figure 17 – collapse mechanism for example platform

ACKNOLLLEDGEMENT
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APPENDIX A
SUMMARY OF APPROACHES, METHODS, OPTIONS, AND PERFORMANCE CRITERIA FOR LIMIT STATE VERIFICATION

<table>
<tr><th>Approach</th><th>Methods include</th><th>Structural Analysis options
(Typical and permitted for design and assessment)</th><th>Performance criteria</th></tr>
<tr><td>Evidence-based</td><td>Engineering Judgement Methods (EJM) for structural system
• as listed in 19901-9 clause 12.3.3</td><td>NA</td><td>comparison</td></tr>
<tr><td>Deterministic</td><td>Safety Factor Methods (SFM) for structural components
(Event RP action effect < representative resistance/ Ω)
• Working Stress Design (eg API RP2A-WSD and API 579) (Note: historically this was Allowable STRESS Design)
• Allowable Strength Design (AISC ASD & ASCE 7-22 cl 1.3.1.2) (action combinations with (typically) variable actions factored in some combinations, and with allowable strength determined by a safety factor)</td><td>typically, linear elastic static or dynamic

NOTE on API 579 levels:
level1= simplified, level2= analytical, level3=numerical</td><td>$\sigma_d(G + Q + E_{100}) \leq \phi F_y$ WSD (RP2A)

$F_d(G + Q + \phi_E E) \leq R_u/\Omega$ ASD (AISC)</td></tr>
<tr><td rowspan="4">Semi-probabilistic</td><td>Partial Factor Methods (PFM) for structural components
(γ_A × event RP action effect< representative resistance/ γ_R)
• ISO 19902, EN1993 & API LRFD & ASCE 7-22 cl 1.3.1.1</td><td>typically, linear elastic static or dynamic</td><td>$S_d(\gamma_G G + \gamma_Q Q + \gamma_E E_{100}) \leq R_d/\gamma_R$ PFM (ISO, EN)
$F_d(\gamma_G G + \gamma_Q Q + \gamma_E E_{100}) \leq \phi_R R_u$ LRFD (AISC)</td></tr>
<tr><td>Event Probability Method (EPM) for structural system
(Event RP action effect < representative resistance)
• $E = E_{10k}$ for L1, $E = E_{1k}$ for L2
• $A = A_{10k}$ for L1, $A = A_{1k}$ for L2</td><td>typically, nonlinear dynamic (NL THA)
may use nonlinear static (eg pushover if no WiD)</td><td>$S_d(E) \leq R_d - S_d(G + Q)$
(E action incrementally applied after G+Q)</td></tr>
<tr><td rowspan="2">Reserve Strength Ratio Method (RSM) for structural system
(100yr action effect × RSR < median resistance)
• RSR magnitude based on partial factors eg ULTIGUIDE 1999, NORSOK N006 cl 8, ISO 19902</td><td>typically, nonlinear static pushover</td><td>$RSR \times S_d(E_{100}) \leq R_m - S_d(G + Q)$</td></tr>
<tr><td>pushdown for gravity critical structures</td><td>$\gamma_{applied} \times \left[\frac{G + \frac{\gamma_Q Q}{\gamma_G}}{\phi_R}\right] > \gamma_G \times \left[\frac{G + \frac{\gamma_Q Q}{\gamma_G}}{\phi_R}\right]$</td></tr>
<tr><td rowspan="3">Reliability-based</td><td>“approximate” Enhanced Action Methods (EAM) for structural system
(Event RP action effect × Cc < median resistance)
• Assessment value method (using design point values)
• Cc (HdF integration) - H and F based on experience (1,3) (ASCE 7-22 clause 1.3.1.3)
• Contour method (similar to environmental contour)</td><td rowspan="3">typically, nonlinear dynamic (NL THA)
may use nonlinear static (eg pushover if no WiD)</td><td rowspan="3">$P_{lse} \leq P_{target}$</td></tr>
<tr><td>Approximate Integration Methods (AIM) for structural components
• FORM
• SORM</td></tr>
<tr><td>Exact Integration Methods (EIM) for structural system
(Integration of failure domain of joint pdf < target probability)
• HdF with H and F based on stratified MCS & simulation (2,3)
• Crude MCS over failure domain of joint pdf (AWARE)
• MCMC sampling by HMC over failure domain of joint pdf
• Directional simulation over failure domain of joint pdf</td></tr>
<tr><td>Risk-informed</td><td>same methods as reliability-based but with explicit consideration of impact (degree of loss) when the relevant limit state is exceeded.

NOTE - SIM EVALUATION determines inspection intervals and anomaly actions using RIDM where likelihood and impact of consequences are typically estimated using engineering judgement or historical performance.</td><td></td><td>$R_{lse} \leq R_{un}$ (4)
$R_{lse} = ALARP$</td></tr>
</table>

1) $H(x)$ typically based on TVM (Tromans, Van der Schuren Method). $F(x)$ based on analytical approximation of aleatory variability.
2) $H(x)$ based on MCS simulation of load from wave events in sampled sea states with sampled crest heights. $F(x)$ based on fit to collapse load simulations (e.g., USFOS)
3) HdF is integration of conditional distributions of the joint pdf.
4) ISO 19901-2 uses this method with Fragility curve $F(x)$ based on experience and H based on simulation.
R_{un} defines the lower bound of the unacceptable annualised risk region of the risk diagram. Performance criteria for each risk type (life-safety [IRPA and SRPA], pollution, and business) are: a) cannot be in the “unacceptable” region of the risk diagram. b) can be in the “broadly acceptable” region of the risk diagram. c) if above the broadly acceptable region but below the unacceptable region, then the risk is tolerable only if it is also ALARP i.e., in the “tolerable-if-ALARP” region of the risk diagram. ALARP is typically demonstrated by CBA (cost benefit analysis).
5) “levels” can be used to identify the approach, unfortunately, levels have different meanings as shown below:

<table>
<tr><th colspan="3">levels</th><th rowspan="2">Approach</th></tr>
<tr><th>API 579</th><th>19901-9</th><th>Reliability theory</th></tr>
<tr><td></td><td>0</td><td>-</td><td>Evidence-based</td></tr>
<tr><td>1
2
3</td><td>(1)</td><td>-</td><td>Deterministic</td></tr>
<tr><td></td><td>1</td><td>I</td><td rowspan="2">Semi-probabilistic</td></tr>
<tr><td></td><td>(1)</td><td rowspan="2">(I)</td></tr>
<tr><td></td><td>2</td><td rowspan="3">Reliability-based</td></tr>
<tr><td></td><td>3</td><td>II</td></tr>
<tr><td></td><td>3</td><td>III</td></tr>
<tr><td></td><td>4</td><td>IV</td><td>Risk-informed</td></tr>
</table>