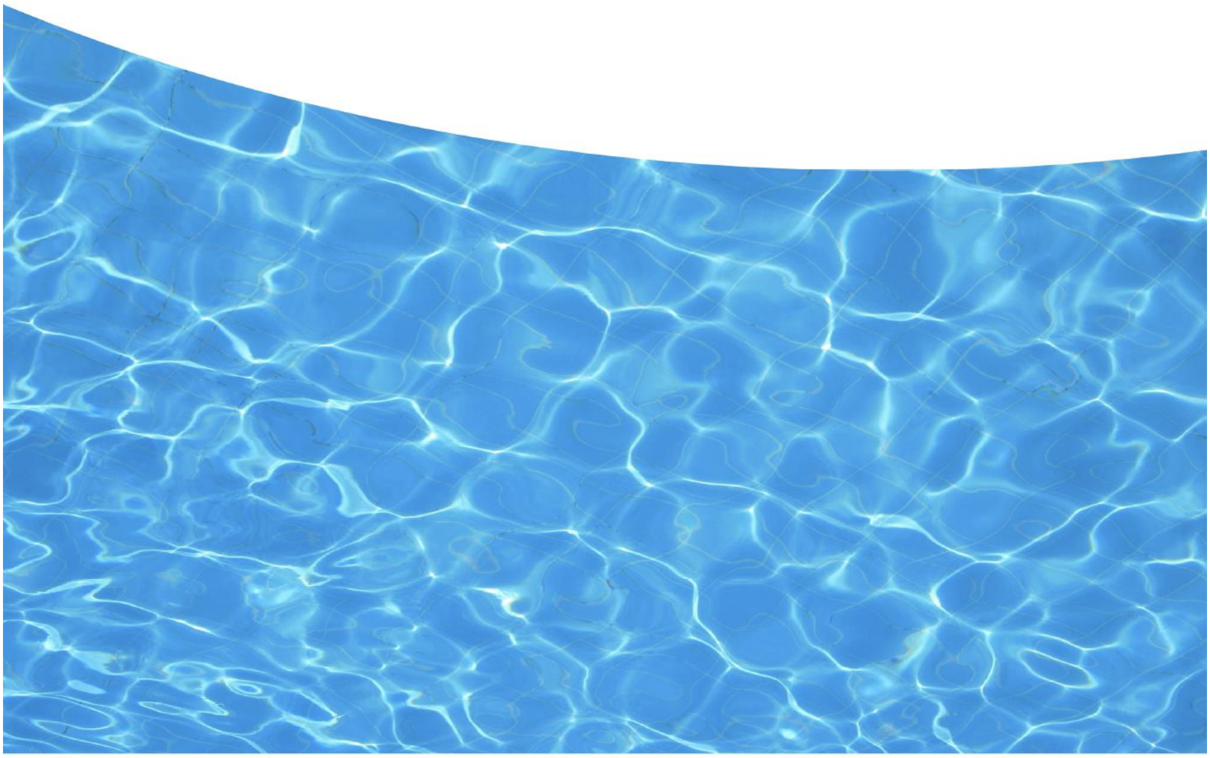
Extreme Environmental Loading of Fixed Offshore Structures: Current Code Requirements

18th October 2018



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Rev	Author	Reviewer	Date
C	Chris Swan	Richard Gibson	18th October 2018
B	Chris Swan	Richard Gibson	4th September 2018
A	Chris Swan	Richard Gibson	26th July 2018

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Revision History

Revision	Description
C	Summary of key points added.
B	Provisional recommendations added.
A	Original report.

Contents

1.0 Summary

This document provides a comprehensive review of the current code requirements as outlined in the ISO, API and NORSOK offshore standards. Whilst there is much in the codes that remains entirely appropriate, the present review has identified other (critical) areas where the recommended practice falls well short of the present state-of-the-art in terms of our physical understanding. The present review has concentrated on these areas. In doing so, it has identified a high degree of commonality between the codes but has also identified areas of significant difference. This is particularly true of the NORSOK standard which raises many more caveats and questions, seeking confirmation that calculation procedures are conservative. In addressing most issues this is achieved through a requirement for extensive model testing. A summary of the key differences between the standards is given in Table 1.1 below. In considering these points it is important to re-state the fundamental differences in the strategies under-pinning these standards; the API approach being intrinsically based upon an exposure matrix category and the process of de-manning.

Issue	ISO	API	NORSOK
General comments	• Not overly prescriptive • Highlights some recent research findings • Inconsistent in the recommended models/procedures	• Very prescriptive • Neglects many recent research findings • Seeks simplification at the cost of accuracy • Introduces unnecessary *ad hoc* procedures	• Less prescriptive than API • Incorporates many recent research findings • Clear in the targets to be achieved • Adopts some state-of-the-art models • Heavy reliance on experimental testing
ULS criteria	• 10^{-2} annual exceedence • Partial factors for load and resistance	• 10^{-2} annual exceedence • Partial factors for load and resistance • Reduced for de-manned structures	• 10^{-2} annual exceedence • Partial factors for load and resistance
ALS criteria	• 10^{-4} annual exceedence • No additional factors	• Not applied	• 10^{-4} annual exceedence • No additional factors
Exposure matrix category	• Not applied	• Well established • Based upon life safety and consequence category	• Not applied
Sub-structure loads	• Morrison's equation • Regular waves and load recipe • Over dependence on base shear and over-turning moment • Validity of recipe uncertain	• Morrison's equation • Regular waves and load recipe • Over dependence on base shear and over-turning moment • Validity of recipe uncertain	• Morrison's equation • Regular waves and load recipe • Over dependence on base shear and over-turning moment • Validity of recipe uncertain

Table 1.1: Summary of the differences between the ISO, API and NORSOK standards for offshore structures.

Issue	ISO	API	NORSOK
Kinematics predictions	• Nonlinear regular waves for sub-structure • Alternatives discussed • No clear guidance provided	• Nonlinear regular waves for sub-structure • Inadequacies described • *Ad hoc* correction factors recommended	• Nonlinear regular waves for sub-structure • Strong caveats provided • Follows closely DNV RP C205 • Second-order irregular wave theory recommended • Mentions fully nonlinear effects • Includes effects of wave breaking • Applied models must be shown to be conservative (laboratory testing)
Importance of WID loads	• Clearly acknowledged • Preference to avoid WID altogether (10^{-4} crest)	• Clearly acknowledged • Based on 10^{-2} crest heights or lower • Possibly of failures acknowledged	• Clearly acknowledged • Preference to avoid WID altogether (10^{-4} crest) • Confirmation based on laboratory testing
Crest elevations	• 10^{-4} conditions drive WID • Forristall (2000) + 15% for area effects • No mentioned of effects beyond 2^{nd} order	• 10^{-2} conditions drive WID • Forristall (2000) + 15% for area effects • No mentioned of effects beyond 2^{nd} order	• 10^{-4} conditions drive WID • Explicit mention of fully nonlinear effects • Forristall (2000) + 10% for area effects and effects beyond 2^{nd}-order
WID calculations	• Recommend silhouette method • Not yet incorporated the “crest conundrum” work	• Silhouette method • Regular waves • “crest conundrum” work adopted • Lots of *ad hoc* corrections • Recipe based on fit to existing WID laboratory data	• Notes silhouette method • Recommends Kaplan et al., (1995) method from DNV RP C205 • Model testing required • Calculations must be shown to be conservative
Slamming loads	• Only locally significant	• Assumed to be primarily vertical • Ignored	• Included • Locally and globally significant • Related to occurrence of wave breaking • Should be assessed experimentally

Table 1.1 (continued): Summary of the differences between the ISO, API and NORSOK standards for offshore structures.

Issue	ISO	API	NORSOK
Wave breaking	• No mention	• No mention	• Specifically included / highlighted • Change in kinematics noted (*u*>1.2*c*) • Investigated experimentally
Model testing	• No mention	• No mention • Calibration of *ad hoc* solutions using historical WID load data	• Extensively included • Key guidance for fully nonlinear problems • Used to ensure calculation procedures are conservative
Long term calculations	• Not mentioned explicitly • Requirements clear • Preferred method of analysis not stated	• Not mentioned explicitly • Requirements clear • Preferred method of analysis not stated	• Method of environmental contour emphasised • Used in conjunction with laboratory testing • Requires extensive validation of calculations • Allows incorporation of highly nonlinear effects • Acknowledges recent scientific advances
Inclusion of recent scientific advances	• Limited • Some acknowledgement • Inconsistent with recommended calculations	• Ignored • Key driver is simplicity • Historical consistency sought	• Most advanced • Key points acknowledged • Fall back is laboratory testing

Table 1.1 (continued): Summary of the differences between the ISO, API and NORSOK standards for offshore structures.

2.0 Introduction: aims and objectives

This document represents the first contribution to objective 1 of the HSE study into extreme environmental loading on fixed offshore platforms. Overall, this objective seeks to review current prediction methods for, and the provision of recommendations on, the effect of extreme environmental loads on the structural integrity of fixed offshore platforms. The work undertaken within this objective is sub-divided into eight parts (labelled SR1-SR8), each with its own specified deliverable. The present document corresponds to the first of these deliverables (O1/D1)[1] and seeks to provide a comprehensive review of the current code requirements for extreme environmental loading in the ISO, API and NORSOK offshore standards; the purpose being to identify the limitations and differences between the recommended approaches.

Within this report we are seeking to identify areas of potential concern, particularly those in which the standards appear to lag behind the present state-of-the-art, both in terms of the adopted models and physical understanding. This does not immediately imply that the standards are inadequate, nor that they produce results that are non-conservative. Indeed, the challenge for future deliverables is to identify when such inadequacies arise and how they can best be addressed. Whilst the review will address all aspects of extreme environmental loading, particular attention will be paid to wave-in-deck loading. This is subject to the largest uncertainty and is widely believed to play a leading role in those instances (worldwide) where structural failure has arisen as a consequence of extreme environmental loading.

The offshore standards included in this review are listed as follows:

(i) ISO 19901-1, ISO 19902 and ISO 19903.
(ii) API RP 2MET, API RP 2A-WSD, API RP 2SIM
(iii) NORSOK N003

In addition, the following clarifications, indications of best practise and design guidance notes are also addressed:

(a) Recommended Practice, DNV-RP-C205
(b) Resolving the API RP 2MET Crest Conundrum

Before outlining and contrasting the approaches adopted in each of the offshore standards, it is important to define some key background material appropriate to the definition of extreme environmental loads. Some of this material is well established in existing (and openly available) technical literature, whilst other material has arisen from recent Joint Industry Projects (JIP's) or is the subject of on-going research. In the latter two cases, this work will be reviewed and explained in subsequent deliverables. However, it is important to provide an overview of the fundamental physics under-pinning the extreme environmental loads at this

[1] Objective 1, Deliverable 1

early stage since this provides the benchmark against which the limitations of the standards can be judged.

Within this deliverable, any limitations will be discussed in a qualitative manner; noting those aspects of the problem that are not addressed in the present standards. The practical significance of these effects, or the flow/loading conditions in which they may become significant and therefore must be addressed, will be dealt with in subsequent deliverables. In effect, this document seeks to highlight those aspects of the present standards which are non-physical, incomplete and inaccurate. In many cases recent research, not least that undertaken within the LOADS JIP, has shown that these short-comings can lead to important changes in the prediction of structural reliability.

3.0 Problem overview

In considering extreme environmental loading, specifically wave loading or (where appropriate) combined wave and current loading, the total global loading can be conveniently divided into two parts:

(i) The total sub-structure load
(ii) The additional wave-in-deck (WID) loading.

If a structure maintains a positive air-gap such that the underside of the deck is located above the highest predicted crest elevation (typically corresponding to an annual exceedence probability of 10^{-4}), the loading associated with (ii) is zero and the maximum total global loading acts on the sub-structure alone. Whilst this may be the case for many structures, it will not be the case for all.

Historically, very few (if any) structures have been designed to withstand WID loading. Rather, they were designed to avoid it; the deck elevation set to maintain a positive air-gap given the regulatory requirements, design conditions and state-of-the-art met-ocean descriptions at the time of the initial design. Unfortunately, many of these criteria have changed such that older structures requiring re-assessment, particularly in respect of life extension, will experience WID loading. The primary reason for this lies in:

(i) The requirement to satisfy updated accidental limit states (ALS) based upon 10^{-4} crest elevations.
(ii) Increased sea state severity, characterised by larger significant wave heights (H_s), arising from improved long-term predictions.
(iii) Larger short-term crest heights due to:
 (a) Nonlinear amplifications arising at third-order and above
 (b) The finite plan area of any topside structure; the corresponding area maximum always being larger than a single-point maximum.
 (c) Nonlinear wave-structure interactions appropriate to large volume structures

(iv) Sea bed settlement leading to reduced deck elevations.

In such cases it is clear that even relatively small levels of deck inundation can result in a very large contribution to the total global loads. The explanation for this lies in two parts. First, the total horizontal forces acting on a relatively large, densely packed (even fully plated) topside structure will increase very rapidly with the level of deck inundation, $\Delta\eta$. Second, in terms of the over-turning moment about the sea bed, the contribution from the WID loads will be large due to the moment-arm effect. Indeed, if one considers the total global load, either base shear or over-turning moment, the inclusion of WID loads produces a step change in the magnitude of the applied loads and a large increase in the variability of the loads. Under these circumstances the accurate prediction of any WID loads becomes fundamental to any reliability calculations.

In considering the prediction of WID loads, the largest crest elevations (responsible for any deck inundation) will be subject to the greatest uncertainty and will also be the most difficult to describe. In addition, the wave properties high in their wave crest, η_c, notably the wave shape, $\eta(x,y,t)$, and the associated water particle kinematics, $u(x,y,z,t)$, are the most difficult to define. Unfortunately, the WID loads are entirely dependent upon these uncertain values, since this is the only part of the wave that enters the deck.

Alongside these difficulties, it is important to remember that the largest waves arising in a real sea state will be unsteady, nonlinear and short-crested. These are fundamental properties reflecting the underlying frequency spectrum, the local wave steepness and the directional spread. Any solution that neglects one or more of these properties has the potential to give very misleading results.

Finally, it is important to understand that there is not necessarily a clear relationship between the 10^{-4} loading event and the load produced by the 10^{-4} wave event. In other words, the largest wave does not necessarily produce the largest loading event. A key part of the explanation for this lies in the associated or local wave period. If this is reduced, the steepness of the wave will increase. With the wave steepness defined in space, effectively representing the gradient of the wave profile, $d\eta/dx$, it is commonly defined by either $½Hk$ or $\eta_c k$; where H is the wave height, η_c the crest elevation and k the wave number, or $2\pi/\lambda$ where λ is the wavelength. The linear dispersion equation linking the wave frequency, $\omega=2\pi/T$ where T is the wave period, and the wave number, k, is defined by:

$$\omega^2 = gktanh(kd), \quad (3.1)$$

where d is the water depth. With a squared relationship between ω and k, a relatively small change in T can have a significant effect on k and hence the wave steepness. Specifically, if both η_c and T reduce below their 10^{-4} predicted values, it is not uncommon for the wave steepness to actually increase. In such cases, the near-surface water particle velocities will be larger, but decay more rapidly with depth. As such, the WID loading will increase and the substructure loads reduce. In effect, changes in the wave steepness can alter the relative importance of the two loading components.

In many sea states these changes will not be large. However, if the sea state is steep and/or attention is focused on the largest/steepest wave events, as would be the case for a 10^{-4} loading event, changes in the wave steepness may involve some degree of wave breaking. This can occur in all water depths. In such cases, changes in both the wave shape and the associated water particle kinematics will be profound; a lower but breaking wave producing much larger near-surface water particle velocities and hence larger local wave loads, particularly WID loads.

Extending this argument it becomes clear that the identification of an appropriate 10^{-4} loading event becomes critically dependent upon the susceptibility of a structure to WID loading and, particularly, the level of wave inundation. As such, two broad categories of problems can be identified:

(i) Structures for which there is little or no risk of WID loading. In these cases design wave conditions that may potentially yield WID loads remain important, but there is no ambiguity about the sea states in which the maximum global loading arises. The largest crest heights will give the largest distributed loading on the sub-structure and also the highest risk of additional WID loading. The only departure from this concerns the distribution of the local loading and the possibility of failure occurring at different elevations above the sea bed.

(ii) Structures for which significant levels of deck inundation can arise. In these cases there is considerable uncertainty as to which wave events will cause the largest loads. The occurrence of breaking waves that are able to penetrate (or break) into a deck structure can profoundly alter the relative importance of the sub-structure and WID loading components. The explanation for this is that wave breaking is associated with substantially increased water particle kinematics high in the wave crest. Moreover, having introduced the possibility of wave breaking, the sea states within which the 10^{-4} loading event occurs also become uncertain. For example, it could be associated with the 3-hour maximum crest elevation within a 10^{-4} sea state. Alternatively, it might be associated with a much rarer individual wave event in a severe but more commonly occurring sea state. In a very simplistic sense, there would be 100 occurrences of a 10^{-2} design sea state in any 10,000 year period. With the 10^{-2} sea state having a smaller spectral peak period when compared to the 10^{-4} sea state, the very largest waves may be steeper and more likely to break. As a result, the 10^{-4} loading event may be due to a smaller but breaking wave.

In considering these two cases it is clear that the occurrence of wave breaking can have a significant effect on the calculated wave loads and hence the structural reliability.

4.0 ISO requirements

This review will begin with the ISO (the International Organisation for Standardisation) requirements concerning extreme environmental loading for offshore structures since these are directly relevant to the UK continental shelf. These requirements are outlined in a number of standards; the present review having considered the following:

ISO 19900	This outlines the general requirements for offshore structures employed within the petroleum and natural gas industries.
ISO 19901-1	This is part 1 of the specific requirements for offshore structures dealing with metocean design and operating considerations. This is the only part of the specific requirements series (parts 1-7) that contributes to the definition of the extreme environmental conditions.
ISO 19902	This concerns fixed steel offshore structures; the implication of the "steel" in the title inferring that the diameters of the individual members, *D*, are small in comparison to the incident wave length, $D/\lambda<0.2$. This defines the so-called slender body regime in which the applied environmental loads can be defined in terms of the incident flow conditions.
ISO 19903	This concerns fixed concrete structures; the concrete construction implying larger diameter members, or large volume structures, for which wave diffraction may play an important role.

Much of the discussion which follows relates to ISO 19901-1 and ISO 19902; although there are immediate implications for ISO 19903. In terms of the general requirements, ISO 19900 divides the limit states into four categories:

(a) Ultimate limit state (ULS)
(b) Serviceability limit state (SLS)
(c) Fatigue limit state (FLS)
(d) Accidental limit state (ALS)

In considering the extreme environmental loads the ULS and ALS conditions are relevant. The ULS condition is defined in terms of a 100-year return period, or an annual exceedence probability of 10^{-2}, and is specified with partial action factors applied to the loads (typically 1.35) and partial resistance factors applied to the structural/material properties; the latter varying from 1.05 to 1.25, but some values extending up to 2.0. In contrast, the ALS condition relates to a 10,000-year return period, or an annual exceedence probability of 10^{-4}, and is applied with no additional factors.

The philosophy under-pinning these two limit states is that a structure should be able to survive the ULS with little or no damage. In contrast, it is accepted that the ALS condition may cause considerable damage, but structural failures causing loss of life and/or major environmental damage are not expected to occur.

The ISO requirements refer to the ULS condition as one involving "*extreme*" environmental actions and the ALS condition as one involving "*abnormal*" actions. The authors of this review do not consider these definitions (*extreme* and *abnormal*) helpful and they will not be considered further.

Interestingly, ISO 19902 suggests that the ALS condition (with all partial action and resistance factors set to 1.0) is less onerous than the ULS condition, provided that sufficient air-gap is available to avoid the occurrence of WID loading. This is an explicit recognition of the danger posed by the occurrence of WID loading. Indeed, in respect of the deck elevation ISO 19902 states in Section A.6.3.3:

> *"A safety margin or air-gap is required between the crest of the design wave and the lowest point (beam, equipment or fixing) of the lowest deck of the platform such that abnormal wave crests do not impinge on the deck. This is necessary, since very large actions can occur if a wave hits the deck. If there is insufficient deck elevation, wave impact can determine the reliability of the structure. Where possible, deck height should be chosen so that the frequency of wave impact on the deck is compatible with the target failure rate of the structure".*

Furthermore, this section also states:

> *"Any determination of the air-gap should account for uncertainty in water depth, structure settlement, sea floor subsidence, sea level rise, storm surge and tide, and abnormal wave crest elevation".*

Whilst the authors of this review agree with these statements, they do not agree with the statement that follows:

> *The deck elevation can be set by either of the following methods:*
>
> *(a) a rational process, using long-term surface elevation statistics and reliability considerations, going to various levels of complexity; or*

(b) experience and judgment, if a rational approach is not possible.

Recent scientific advances are such that a "*rational approach*" is both possible and essential. Most importantly, "*experience and judgement*" should not be used as an excuse to do what has historically been done, not least because the goal posts have shifted in the sense that recent scientific advances suggest that, in many circumstances, the 10^{-4} ALS crest heights have increased and the role of wave breaking is much more significant.

ISO 19902 also acknowledges that, where data is both available and sufficient, the 100-year return period (ULS) may be applied to the responses (action effects) of the structure instead of to the metocean design parameters. This acknowledges that the largest crest elevations do not necessarily produce the largest responses.

Both ISO 19901-1 and ISO 19902 note that the ULS environmental loads will be caused by the combined action of waves, currents and wind. Indeed, they propose one of three methods to define the ULS condition:

(a) 100 year return period wave height (significant or individual) with associated wave period, wind and current velocities;

(b) 100 year return period wave height and period combined with the 100 year return period wind speed and the 100 year return period current velocity, all determined by extrapolation of the individual parameters considered independently;

(c) any reasonable combination of wave height and period, wind speed and current velocity that results in:

- *the global extreme environmental action on the structure with a return period of 100 years, or*
- *a relevant action effect (global response) of the structure (e.g. base shear or overturning moment) with a return period of 100 years.*

Whilst (a) is commonly applied and (b) will undoubtedly lead to a degree of conservatism, (c) is the preferred approach and consistent with the comments noted above. However, when considering the global response of a structure ISO 19902 is overly reliant on an assessment of the base shear and over-turning moment. In practice, the global failure of a structure[2], involving the complete loss of structural integrity, can occur at any elevation above the sea bed. The challenge as far as structural reliability is concerned is to identify the elevation at which failure is most likely to occur, given the full range of possible loading events. This will be dependent upon both the layout of the structure and the distribution of the applied loads; the latter being critically dependent upon the form of the incident waves.

[2] In discussing this point it is important to distinguish between a global and local failure; the latter involving damage to or loss of an individual member (or members) without suffering the catastrophic loss of the entire structure.

Setting aside concerns regarding the appropriateness of the base shear and over-turning moments as an indicative measure of the applied loads, ISO 19902 notes that the ULS global actions can be calculated by the vector sum of:

(a) local hydrodynamic drag and inertia actions due to waves and currents integrated over the whole structure,
(b) dynamic amplification of wave and current actions, and
(c) actions on the structure and the topsides caused by wind.

This approach pre-supposes that the 10^{-2} ULS crest elevation causes no WID loads. If this is not the case, ISO 19902 suggests that the additional loads should be added to (a).

Having acknowledged the need to identify the environmental conditions (notably the waves) consistent with the 10^{-2} action effect, the methodology outlined above is appropriate. However, important concerns arise in the details of the recommended calculations. To be fair, both ISO 19901-1 and ISO 19902 go some way in highlighting some of the important physical effects, but then make recommendations concerning the application of inappropriate and inaccurate models. Whilst there are caveats attached to the accuracy of some of these models, the very fact that they are mentioned encourages their use. As a result, the ISO documents are not indicative of best practice. Splitting the load calculation problem into its two components (sub-structure and WID) the primary areas of concern are listed as follows.

4.1 Sub-structure loads

In terms of calculating the sub-structure loads, ISO 19902 proposes a straightforward application of Morison's equation, integrated over all members, to define the global loads. Recommendations are made concerning the hydrodynamic loading coefficients, C_d and C_m; where C_d is the drag coefficient and C_m the inertia coefficient. Indeed, a range of values are given depending on whether the surface of the body is smooth or rough (Table 9.5.1), with additional figures given in Annex A9 defining:

- C_d as a function of the relative surface roughness
- Wake amplification factors for C_d
- C_m as a function of the Keulegan-Carpenter number, KC
- C_m as a function of KC/C_{ds}, where C_{ds} is the steady flow drag coefficient

Whilst this information is all highly relevant (and appropriate) the success of Morison's equation is dependent upon both an accurate choice of loading coefficients and the accurate prediction of the water particle kinematics; the latter primarily relating to the wave motion, but also including combined wave-current conditions.

In considering the kinematics predictions, ISO 19901-1 correctly notes that ocean waves are irregular (unsteady), directional, and (in the case of the largest waves) nonlinear. However, it goes on to add that:

> "*in some applications periodic or regular waves can be used as an adequate abstraction of a real sea for design purposes*".

Furthermore, it also notes:

> "*periodic waves are the building blocks for the linear random wave model*".

Whilst the latter is undoubtedly true, it is not a justification for the use of a periodic wave model. After all, the waves of interest are neither periodic nor linear

To confuse the matter further, ISO 19901-1 goes on to state that where closer agreement to real waves is required, fully nonlinear wave models are available but require careful calibration to achieve consistent reliability levels. The present authors would argue that for both the ULS and, in particular, the ALS condition close agreement to "real waves" is essential and no calibration should be required. Indeed, this is the whole purpose of applying a more complex, more accurate, wave model.

In truth, the discussion of wave modelling within IOS 19901-1 is both inconsistent and miss-leading. On the one hand it readily acknowledges the (indisputable) advantages of adopting fully nonlinear wave models. On the other hand, it keeps drawing the reader back to simplistic and inappropriate wave models whose sole advantage is the perception that that they are easier to apply, irrespective of the fact that they give the wrong solution. Or, to be fair, they give a correct solution to the wrong problem.

A clear example of this is given at the end of Section 8 and the beginning of Section 9. In Section 8.3.4 headed: "wave kinematics - velocities and accelerations" it first notes that linear wave theory is a first order approximation and, as such, only valid for waves of low steepness. This directly implies that it is not relevant for the description of either the ULS or the ALS condition. Nevertheless, it continues to discuss the advantages of stretched linear random wave theories (specifically Wheeler stretching and Delta stretching), but omits to make it clear that these are also linear solutions. However, it does conclude that these models can be non-conservative and should "only be used where their application can be justified". More importantly, it notes that:

> "*Better approximations are recommended and are provided by nonlinear wave models*".

This statement directly refers to Section 8.6 (of ISO 19901-1) in which it is further noted:

> "*Neither regular (of any order) nor linear random wave theories give the correct elevation of the wave crest or kinematics over the water column*"

However, as an alternative it suggests that second-order random wave theory provides an adequate approximation for "moderately steep waves". Unfortunately, in most practical circumstances neither the ULS nor the ALS design wave conditions will be moderately steep. These are both extreme waves, with very small annual exceedence probabilities. Indeed, this is readily acknowledged in the text (Section 8.6) which states:

> "*Examples of where a second-order spectral model can be inaccurate and where a fully nonlinear model is a better choice include the calculation of loads during the inundation of the deck of a platform by green water and the calculation of wave forces on a fixed structure in very steep waves*".

Unfortunately, these are exactly the calculations that are required in respect of the ULS and ALS conditions.

More worryingly, ISO sets aside an entire section (8.4) to a discussion of regular waves, despite having noted (see above) that they do not give correct descriptions of either the crest elevation or the kinematics. To avoid a direct contradiction Section 8.4.1 notes:

> *"For determination of actions by individual waves on structures, a nonlinear periodic wave theory may be used with a calibrated loading recipe",*

and adds:

> "Calibrated loading recipes for drag-dominated structures are coded in typical loading software".

This final statement is particularly troubling because it is unclear exactly how such a calibration has been achieved. First, it cannot have been produced on the basis of laboratory observations since these would require Reynolds number similarity and this cannot be produced in even the largest wave basins. Second, whilst there are undoubtedly some records of the loading experienced by steel jacket structures due to large waves, very few of these cases approach conditions relevant to ULS conditions, and none are consistent with ALS conditions. Historically, the simultaneous measurements of water surface elevation and total global load recorded at Shell's Tern platform in the North Sea are amongst the most useful field data since it includes a small number of large wave events. However, the largest reliable wave record has a wave height of H=21.5m and a local steepness of $\frac{1}{2}Hk$=0.24. This falls well short of the ALS conditions for the North Sea which has a steepness in the order of $\frac{1}{2}Hk$=0.30. Without relevant field data, recorded at multiple locations using different measuring systems, it is very difficult to see how the effective calibration of a loading recipe, to account for the acknowledged inadequacies of the recommended wave model, can be achieved.

In summary, the calculation of sub-structure loads appropriate to ULS and ALS conditions outlined in ISO 19902 is based upon a well-established load model, with realistic loading coefficients, and gives important practical guidance. However, the recommended solutions for the prediction of the water particle kinematics, both velocities and accelerations, fall well short of the present state-of-the-art.

4.2 Wave-in-deck (WID) loads

In respect of new platform designs, ISO 19902 is clear that the deck elevation should be set to avoid the occurrence of WID loads over the operating life of the structure. However, it is equally clear that there are a significant number of existing structures which will be subject to WID loading, particularly in respect of their ALS condition. It is in seeking to address this problem that IOS 19902 is at its weakest. To be fair it does clearly state that the calculation of WID loads is: “an area of on-going research”. Whilst this is certainly true, neither ISO 19901-1 nor ISO 19902 are indicative of present best practice. The explanation for this lies in two parts.

First, the criticisms of the proposed wave modelling (noted in section 3.1 above) become critically important in respect of WID loading; the problem being governed by the crest elevation, the wave shape and the near-surface water particle velocities. This is readily acknowledged in Section 8.6 of ISO 19901-1 with “fully nonlinear wave models” being the preferred choice for WID load calculations.

The explanation for this lies in the fact that wave nonlinearity arises from the two nonlinear free surface boundary conditions. As the source of this nonlinearity is approached, the instantaneous water surface and, specifically, the wave crest, the relative importance of this nonlinearity increases. In respect of WID loading, it is only the very upper levels of the wave crest, close to the water surface, that enter a deck structure and define the applied loads. It therefore follows that the wave nonlinearity is fundamental to the definition of both the occurrence and magnitude of any WID loading.

Second, ISO 19902 outlines a simple Silhouette method to estimate the global WID loads. This method is described in some detail in Annex A Section 24.7.3. In essence, the method defines a drag type loading event in which the WID load (or action) due to combined waves and currents is expressed as:

$$F_{WID} = ½\rho_w C_d(\alpha_{wk}u_w + \alpha_{cb}U_c)^2 A, \tag{4.1}$$

where ρ_w is the density of water, A is the Silhouette or inundated area, u_w is the maximum wave-induced horizontal fluid velocity, α_{wk} a kinematics factor to account for directionality, U_c the current velocity in line with the wave, α_{cb} the current blockage factor and C_d an inferred ‘drag coefficient’ calibrated to wave tank tests of “quite simple” topside structures. Guidance is provided concerning the variation of C_d with the deck type (heavily equipped [solid], moderately equipped and bare [no equipment]) and with the orientation of the structure relative to the mean wave direction (θ_w=0° [end-on or broadside] and θ_w=45°).

The output from this model is a maximum WID load (not the corresponding time-history), together with guidance as to the elevation at which it should be applied; the latter being specified as 50% of the distance between the lowest point of the silhouette area and the lower of either the wave crest or the top of the silhouette area. With the magnitude and position of the WID load described, this is added to the maximum global sub-structure load to describe the total base shear and over-turning moment in accordance with Section 9.4.3.

There are three important limitations to this approach:

(a) With no information describing the phasing of the maximum sub-structure load and the maximum WID load, there is little choice but to assume that they are co-incident and therefore additive. This will inevitably introduce a degree of conservatism.
(b) In the absence of a WID load time-history, $F_{WID}(t)$, a dynamic analysis, even in its simplest form, cannot be undertaken.
(c) The calibration of the model has historically been limited to one laboratory data base involving relatively simple topside structures. Importantly, this data base specifically avoids all occurrences of wave breaking. Given the recent advances reported by Latheef & Swan (2013), Karmpadakis et al (2019), Karmpadakis & Swan (2020) and Ma & Swan (2020a,b), this is clearly an important omission.

In describing the silhouette method ISO 19901-2 emphasises its simplicity and notes that the variation of the WID load for a given wave height is "rather large"; the coefficient of variation (COV) estimated to be 0.35. It also notes that WID loading is complex and influenced by factors including:

(1) the type of waves,
(2) level of inundation,
(3) deck configuration,
(4) wave phase,
(5) deck proportions.

In considering (1) it is unclear if this relates to the nature (or limitations) of the applied wave models or the physical properties of the actual waves causing the WID load. If the latter, then this may be the first implicit reference to the importance of wave breaking (see below).

In respect of WID loading ISO 19902 concludes that the simplified silhouette model should only be used when it can be shown to be appropriate, although it is unclear how this would be achieved. Furthermore, it also notes that for a more accurate analysis specific attention should be given to the predicted kinematics. Again, no further information is provided in this regard.

In considering the setting of deck elevations, ISO 19901-1 suggests that crest elevations can be satisfactorily predicted using a second-order random wave model; the two parameter Weibul fit to second-order crest elevations described by Forristall (2000) being outlined as the preferred method. No assessment is made of effects arising beyond second-order. In addition, it notes that corrections need to be applied to account for the finite area of a deck structure. This follows the work of Forristall (2007), acknowledges that the probability of experiencing a given crest elevation over a representative plan area is substantially larger than the usual point statistics, and suggests that the maximum crest elevation for a given exceedence probability may be increased by as much as 15% due to the area correction. Whilst this effect is undoubtedly important, recent research suggests that a 15% increase in the largest crest elevation due to area effects alone is excessive.

ISO 19901-1 addresses two other issues that are important for the calculation of WID loads, although they are not directly mentioned in this context. First, it correctly notes that in the case of combined waves and currents, the co-existence of the two leads to a stretching and compression of the current under the wave crests and troughs respectively. In respect of WID loading only the wave crest is of interest and, in this case, the divergence of the streamlines will lead to a reduction in the local current velocity. This is not correctly noted in either of the procedures given in ISO 19901-1 leading to conservative predictions of the combined velocities and hence an over-prediction of the WID loads. Given that the latter is dependent on the square of the former, this effect can be substantial.

The second effect concerns the occurrence of wave slamming (or slapping) on structural members high in the wave field. ISO 19902 suggests that these can be neglected as they are only of "*local significance*". Whilst they are certainly important in respect of local loading and possible member failure, they may also have a global significance, particularly if they contribute to the onset of a dynamic response. For example, impact loads may well occur on the main deck beams, providing an important contribution to the total WID load. Furthermore, if the platform incorporates a sizable and densely packed riser/conductor array, the slam loads experienced by these elements can be significant, both in terms of local and global damage.

Finally, despite emphasising the importance of WID loading, particularly in the context of ALS conditions, ISO 19902 makes no mention of wave breaking. Moreover, ISO 19901-1 only mentions wave breaking in the context of depth-limited shallow water conditions and in respect of wave shoaling. The only exception to this concerns a comment in Section A8.6 in which it is noted that Fourier based nonlinear wave models are limited in their application when a wave becomes very steep-fronted at the onset of breaking.

Interestingly, there is no discussion as to the importance of deep water wave breaking and the implications that this has for the extreme wave loads (both ULS and ALS) and hence the structural reliability. This is despite the fact that:

(a) There is a growing awareness from field observations in both intermediate and deep water that wave breaking is an important feature of most extreme sea states, particularly those that are relevant to design/re-assessment.
(b) Observations of long random wave records in highly controlled laboratory conditions confirm that many of the largest and steepest wave events exhibit some form of wave breaking, either spilling of varying intensity or full over-turning. Moreover, when WID load testing is undertaken, many of the largest individual loading events are associated with breaking waves.

In summarising the treatment of WID loads within the relevant ISO documents, it is clear that whilst the importance and complexity of these loading events is clearly stated, neither the proposed wave modelling, the method of load calculation, nor the calculation of reliabilities in terms of pre-determined wave events is adequate. Moreover, the proposed approaches do not represent present best practice and are certainly not based upon the present state-of-the-art.

5.0 API requirements

The American Petroleum Institute (API) standards relating to fixed offshore structures are sub-divided into three parts:

(i)	API RP 32MET: 2014	dealing with metocean conditions
(ii)	API RP 2A-WSD: 2014	addressing the design of new structures using a working stress design approach
(iii)	API RP 2SIM:2014	addressing structural integrity management and related to the re-assessment of existing structures.

In considering these documents it is relevant to note that API RP 2A-LRFD dealing with load and resistance factor design has been withdrawn and will not be considered herein.

Taken as a whole, these documents exhibit many similarities with their ISO equivalents (Section 3.0). Indeed, API RP 2MET: 2014 is also listed as ISO 19901-1: 2005 (modified), suggesting that there was convergence between API and ISO, at least in respect of metocean conditions. However, it is our understanding that although there was a clear intention for API to adopt the latest ISO standard (ISO 19901-1: 2015) this has not yet happened and is unlikely to occur in the near future. Nevertheless, the similarities between ISO and API are such that many of the short-comings noted in Section 3.0 are also relevant to the API standards. However, the API documents set out to be more prescriptive, setting out specific models that should be applied and how. As a result of this philosophy there are several instances where inappropriate and inaccurate models and procedures are recommended, necessitating the introduction of *ad hoc* and non-physical corrections to account for well-established and well known modelling errors. Moreover, with a desire to be prescriptive and in seeking to keep procedures as simple as possible, short-cuts are recommended (or imposed) that reduce the accuracy of the desired reliability calculations. Taking each of these documents in turn, comments are given as follows:

5.1 API RP 2MET: 2014

This document sets out to define the procedures and metocean parameters appropriate to US waters, with a particular emphasis on different sectors of the Gulf of Mexico (GoM). Throughout the suite of API standards it is clear that the recommended extreme relates to an event with an annual exceedence probability of 10^{-2} (page 3; Section 3.6 and many other places). The justification for this is based upon experience with major platforms in the GoM.

API RP 2MET: 2014 (hereafter referred to as 2MET) restates the three methods of defining an environment that generates the extreme direct action and, generally, also the extreme action effect as outlined in (a), (b) and (c) in Section 3.0, page 8 of this document. Within 2MET a more detailed explanation of the 3 cases is provided and, more importantly, it is clearly stated that: “when sufficient data are available method (c) should be used”. This is the method that adopts any reasonable combination of wave height and period, wind speed and current velocity that results in either:

(i) The extreme environmental action on the structure with a specified return period, or, more importantly,
(ii) The extreme action effect (global response) of the structure (base shear or over-turning moment) with a specified return period.

With a clear understanding that (ii) is what is required, 2MET returns attention to the appropriateness of a deterministic design wave arguing that:

"In some applications, periodic or regular waves can be used as an adequate abstract of a real sea for design purposes".

Moreover, it goes on to state that the relevant applications are those cases in which the action effect(s) are quasi-statically related to the associated wave actions and specifies the design wave in terms of the 10^{-2} wave height (H_{max}) and an associated wave period (T).

In adopting this approach, three facts are important:

(a) In respect of extreme wave conditions the *n*-year maximum crest elevation, $\eta_{c\ max}$, does not necessarily occur together with the *n*-year maximum wave height, H_{max}.
(b) Irrespective of whether interest lies in the prediction of sub-structure or WID loads, it is widely acknowledged that the maximum loads (or action effects) will in many cases be associated with the largest crest elevations, $\eta_{c\ max}$. This is particularly true in respect of WID loading since it is the crest of the wave that enters the deck and causes the load.
(c) Successive API documents have been quick to acknowledge that most historical platform failures have been attributed to waves impacting the platform deck, thereby resulting in a large stepwise increase in the fluid loading.

Given these facts if a design wave is to be expressed in terms of metocean parameters rather than loads (or action effects) an obvious choice would be the *n*-year crest elevation. However, because the API standards have resisted any move away from all but the simplest regular wave theories (Stokes 5^{th}-order or steam function solutions), despite their well-documented failings when it comes to the description of large ocean waves, and because these inappropriate wave models are typically expressed in terms of the wave height, H, the API approach remains one based upon the *n*-year wave height. This greatly complicates the prediction of WID loads, particularly in respect of their return periods, despite the acknowledgement that they are the most common cause of failure.

Interestingly, although this difficulty arises as a direct consequence of the modelling procedures adopted, it is readily acknowledged at an early stage of the 2MET document; Section 3.4 (page 2) noting that:

> *"Because of the simplified nature of the models used to estimate the kinematics of the design wave, the design crest elevation can be different from, usually somewhat greater than, the crest elevation of the design wave used to calculate actions on the structure".*

Having acknowledged that WID loading is the most common cause of failure for GoM structures, 2MET becomes surprisingly inconsistent. Examples of this are given below. In each case the inconsistencies are driven by a reluctance to move away from deterministic regular wave solutions.

Example 1: Having noted the importance of WID loading in respect of platform failure, Section 8.8 of 2MET correctly notes that the distribution of extreme crest elevations is required for setting minimum deck heights on bottom founded structures and for assessing the probability of green water intruding onto the topsides of all types of structures. This is entirely correct, but half a page above in Section 8.5 it notes that the maximum height of an individual wave with the specified return period is the single most important wave parameter to be determined. Moreover, it notes the required long-term individual wave height, H_N, should be established by convolution of the long-term distributions derived from the data with a short-term distribution that accounts for the distribution of individual wave heights in a sea state. The present authors fully support this approach but, given the importance of WID loading, believe the distribution of crest heights is more important than wave heights.

Example 2: In respect of water depth and wave breaking, Section A.6.2 correctly notes that in shallow water the empirical limit of the wave height is approximately 0.78 times the local water depth for waves that are long-crested. For short-crested waves the wave height can approach 0.9 times the local water depth and the breaker height also depends on beach slope. In deep water it also notes that waves can break with a theoretical limiting steepness of 1/7. This latter value is based on regular/steady waves and is therefore not appropriate to realistic random waves. More importantly, 2MET makes no attempt to explain how such waves should be modelled, despite noting the importance of breaking in shallow water. Furthermore, it notes that shoaling and non-linear processes affect crest elevations as waves move into shallow water; the proportion of the wave height above the nominal still water increasing as the water becomes shallower. However, it omits to note that in deeper water nonlinearity drives a similar change in the crest trough asymmetry and that this cannot be modelled by regular waves.

Example 3: In respect of nonlinearity 2MET addresses the accuracy of both hindcast models and measured data, arguing that:

> "*Wave models are only as good as the physics that are incorporated in them. The strengths and weaknesses of any particular data set should be recognized throughout the process of its analysis and interpretation*".

This is very good advice, but conflicts with the fact that earlier in the main body of the report (Section 8.1) it first notes that ocean waves are:

> "*irregular in shape, vary in height, length and speed of propagation, and approach a structure from one or more directions simultaneously*".

However, it then argues that a regular wave theory can be "adequate" to define design waves despite the fact that they are regular, of constant height, length and speed of propagation and uni-directional.

Example 4: In respect of extreme waves 2MET notes that other data that should be developed (presumably because it describes an important effect) includes:

(a) the probable range and distribution of wave periods
(b) the wave energy spectrum in the sea state.

Unfortunately, a steady design wave solution is based upon a single wave period and no distribution of free wave energy.

Example 5: In Section A.8.2 2MET correctly notes that:

> "*the highest action, or the largest action effect on, a structure is not necessarily induced by the highest sea state or the highest wave in a sea state. This is due to the nature of wave action and the sensitivity of structures to the frequency content of waves in a sea state".*

This is entirely correct and emphasises the importance of the fact that the frequency content cannot be modelled by regular waves. Moreover, the most likely explanation for this statement lies in the nature of the wave action, particularly the occurrence of wave breaking, and this is simply not addressed.

Example 6: In a discussion of two-dimensional wave kinematics (Section A.8.4), 2MET correctly notes that regular wave models:

> "All compute a waveform that is symmetric about the crest and propagates without changing shape".

It has already been noted that neither of these characteristics are representative of real ocean waves. Furthermore, it goes on to note that:

> "*The theories differ in their functional formulation and in the degree to which they satisfy the*
> *nonlinear kinematic and dynamic boundary conditions at the wave surface*".

Indeed, it argues that the selection of the most appropriate theory should be based on the accuracy of this fit. Unfortunately, the extent to which the nonlinear boundary conditions are satisfied on an unrealistic and unrepresentative regular wave profile is no guarantee of its ability to describe the water particle kinematics associated with an unsteady, irregular and non-symmetric wave event. Indeed, the only requirement that matters is the ability to satisfy these boundary conditions on the actual (unsteady) wave profile. Clearly, this cannot be achieved by a regular wave model, of any order.

Example 7: To be fair, 2MET does make mention of the so-called NewWave theory. This is based upon statistical theory first proposed by Lindgren (1970), Boccotti (1983) and, more recently, by Tromans et al (1990); the latter contribution introducing the name. However, whilst noting that this is an unsteady wave model, it omits to note that it is a linear model and, therefore, not appropriate to the description of design waves without the inclusion of nonlinear corrections. Moreover, 2MET fails to note that if such corrections are applied, the need to apply any form of stretching (Delta stretching) is eliminated and an important source of inaccuracy removed.

However, 2MET goes on to suggest that with the application of the NewWave model:

> "*the kinematics are evaluated at only one instant during the wave evolution and then frozen as the wave propagates through the structure*".

This is wholly unnecessary and, in effect, seeks to turn the NewWave solution into a quasi-steady state model. In so doing, many of the advantages of this model are lost, particularly in respect of predicting the WID loads on a topside structure with a significant plan area.

Having raised these issues, it is important to note that the authors of 2MET appear to be aware of these inconsistencies but have taken the view that ease of use and historical consistency (or a reluctance to change) are more important than technical rigor. This is further examined in Section 4.1.1 which provides a brief review of a recent contribution concerning the so-called "crest conundrum" arising as a result of the procedures recommended within 2MET. Unfortunately, the approach that has been adopted involves further *ad hoc* corrections rather than the application of rigorous models that would avoid the problem in the first place.

5.1.1 Resolving the API RP 2MET Crest Conundrum

This so-called conundrum relates to a mismatch in the description of the design crest elevation. It has already been noted that the crest elevation corresponding to the *N*-year wave height does not correspond to the *N*-year crest elevation. Moreover, if the *N*-year wave height is modelled by a non-linear regular wave model (either a Stokes 5th-order solution or a stream function solution) the corresponding crest height will be substantially lower in deep water. This defines what a recent API working group has referred to as the "negative conundrum"; the design predicted crest height being lower than the actual crest height for the same return period.

This has significant implications for the predicted WID loads. In an attempt to correct for this, thereby achieving the best possible prediction of the WID loads using a design regular wave and a calibrated silhouette method, the working group considered four options:

(a) To reduce the modelled deck elevation maintaining the actual level of wave inundation.
(b) To compute the sub-structure and WID loads separately; the latter being solely dependent upon the crest kinematics assuming the silhouette method is employed.
(c) To increase the design wave height until the corresponding crest elevation matches the *N*-year crest elevation.
(d) Increase the water depth until the crest elevation associated with the *N*-year wave height (modelled using a regular wave) matches the *N*-year crest elevation.

Having considered all four options, the working group adopted (d). However, this has un-intended consequences in that an increase in the water depth produces a reduction in both the water particle velocities and the accelerations in a regular wave solution. This is problematic because in the original calibration of the silhouette model for WID loading, Finnigan (1991) showed that the best fit to the measured WID load data, or the fit with the least scatter having defined an optimal drag coefficient, was achieved when the crest elevation of the design wave event was closest to the actual crest elevation recorded in the random wave tests. This approach is much closer to method (c) above and confirms the importance of the kinematic predictions when it comes to WID load predictions.

In an attempt to get both the level of wave inundation and the wave kinematics approximately correct, at least as far as the inputs to the simplified WID load model are concerned, the working group proposed continuing with a deterministic design wave modelled on the basis of a regular wave specified in terms of a *N*-year wave height but with the following changes:

(i) Change the water depth to give the correct *N*-year crest elevation and hence the correct inundation.

(ii) Adjust the period of the wave so that the predicted kinematics are closely aligned with those that would have been predicted had the water depth been held constant and the wave height increased until the desired crest elevation was matched.

In practice, these *ad hoc* changes need to be undertaken simultaneously since (ii) also effects the wave nonlinearity and therefore the predicted crest elevation and hence the degree of inundation. As a result, the two *ad hoc* changes need to be undertaken iteratively. To provide guidance in this regard, Santala (2017) gives tables of correction factors for typical GoM conditions.

To summarise this approach an inappropriate regular wave model remains the basis for the predicted WID loads. This is based upon an *N*-year wave height, in circumstances where the loads are fundamentally driven by crest heights. To adjust for this, *ad hoc* corrections are made to both the water depth and the wave period in an attempt to facilitate the use of a simplified Silhouette method and to take advantage of an existing calibration.

Further details of the approach are given in Santale (2017). This also notes that the solution will be incorporated in the next revision of RP-2MET expected in 2018.

5.2 API RP 2A-WSD: 2014

The 22nd edition of this document, hereafter referred to as 2A-WSD, outlines best practice across a very broad range of disciplines relevant to the design of offshore structures. This review will concentrate on those areas which contribute most to the uncertainty in the calculated reliability. Given the present state-of-the-art this inevitably concentrates on the "loading" rather than the resistance side of the equation. As such, many of the issues are related to the inputs that arise from 2MET and their likely consequences in terms of 2A-WSD.

Importantly, 2A-WSD readily acknowledges the significance of WID loading. In Section 4.2.7 on deck elevation, it states:

> "*Unless the platform has been designed to resist these forces, the elevation of the deck should be established to provide adequate clearance above the design maximum crest elevation. Consideration should be given to providing an "air-gap" and an additional*

allowance for local maximum crest elevations, which are higher than the design maximum crest elevation".

As stated previously, the API definition of an extreme is based upon a 100-year event or one with an annual exceedence probability of 10^{-2}. In respect of this extreme, 2A-WSD is very specific:

"*This is applicable only to new and relocated platforms that are manned during the design event or that are structures where the loss of or severe damage to the structure could result in a high consequence of failure. Consideration may be given to reduced design requirements for the design or relocation of other structures that are unmanned or evacuated during the design event and have either a shorter design life than the typical 20 years or where the loss of or severe damage to the structure would not result in a high consequence of failure*".

To be clear, 2A-WSD also states:

"*However, not less than 100-year oceanographic design criteria shall be considered where the design event may occur without warning while the platform is manned and/or when there are restrictions on the speed of personnel removal (e.g. great flying distances)*".

API has long maintained a Life Safety Category (S-1, S-2, and S-3) and a Consequence Category (C-1, C-2, and C-3) which together define an Exposure Category Matrix (L-1, L-2, and L-3). Details of these are given on Table 4.1 below.

Life Safety Category	**Consequence Category**		
	C-1, High Consequence	C-2, Medium Consequence	C-3, Low Consequence
S-1 manned-nonevacuated	L-1	L-1	L-1
S-2 manned-evacuated	L-1	L-2	L-2
S-3 unmanned	L-1	L-2	L-3

Table 5.1: Exposure Category Matrix (copied from Table 4.1 page 18, 2A-WSD)

Using these definitions 2A-WSD provides two further clarifications:

"*For all new Category L-1 structures located in U.S. waters, the use of nominal 100-year return period is recommended. For all new Category L-2 and L-3 structures located in the U.S. Gulf of Mexico north of 27 °N latitude and west of 86 °W longitude, guidelines for using shorter return criteria are provided*".

and:

> "*Manned-nonevacuated platforms are presently not applicable to the U.S. GoM waters where platforms are normally evacuated ahead of hurricane events. The metocean design criteria in Section 5 have not been verified as adequate for manned-nonevacuated platforms in the U.S. GoM*".

This serves as a footnote to the Exposure Category Matrix given in Table 4.1 of 2A-WSD.

These requirements, particularly the acceptance of design conditions below the 10^{-2} condition, are different to those adopted in ISO (Section 3) and NORSOK (Section 5); the justification being a strict imposition of evacuation for all manned platforms in the GoM.

In respect of the water particle kinematics 2A-WSD stays firmly wedded to a design wave event based upon a regular wave theory, arguing that in many cases a Stokes 5^{th}-order wave theory produces acceptable results. Moreover, it notes (in Section 5.3.1.2.4) that "real world" waves are directionally spread and that this should be modelled by a wave kinematics factor lying in the range 0.85-0.95 for tropical storms and 0.95-1.0 for extra-tropical storms. This is consistent with ISO 19901-1. However, 2A-WSD goes one step further and notes (in Section B5.5.1.2.4) that real waves are irregular commenting that:

"*If an 'irregularity factor' less than unity is supported by high quality wave kinematics data, including measurements in the crest region above mean water level, appropriate for the types of design-level sea states that the platform may experience, then the 'spreading factor' can be multiplied by the 'irregularity factor' to get an overall reduction factor for horizontal velocity and acceleration*".

Interestingly, it does not say whether an irregularity factor >1.0 should also be applied above SWL if the data suggest it. In considering the concept of an overall reduction factor, suggesting that predicted velocities high in the wave crest might be substantially less than 85% of the Stokes' predicted velocities, even in those cases where a local wave period would imply a high level of wave breaking, is concerning. Indeed, the present state-of-the-art suggests that this would be non-conservative, both in terms of velocity predictions and, more importantly, the associated loads.

Finally, in considering the global structural loads (Section 5.3.1.2.12) 2A-WSD suggests that:

> "*Local slamming forces can be neglected because they are primarily vertical*".

This is incorrect. Two examples where such forces can be large and horizontal include:

(i) The slamming forces arising high in a structure due to wave breaking.
(ii) The forces acting on the main deck beams.

Unfortunately, there is no discussion of wave breaking and the action effects arising in 2A_WSD.

5.3 API RP 2SIM: 2014

This recommended practice, hereafter referred to as 2SIM, concerns the implementation and delivery of a process to manage the structural integrity of existing fixed offshore structures. It considers a very broad range of activities from surveys, to damage evaluation, to risk reduction and platform decommissioning. In the present context, and given earlier comments, the present review is particularly interested in the assessment of metocean loading appropriate to existing structures.

To demonstrate that a platform located in the GoM is fit-for-purpose, 2SIM provides a revised set of metocean criteria/loads. However, it is noted that the adoption of these values may leave a platform that is vulnerable to damage or collapse in a hurricane, particularly for L-2 or L-3 exposure category platforms designed prior to the 20th edition of API RP 2A-WSD. This is justified on the basis that the assessment approach is structured so that the damage to, or collapse of, a platform will not increase life safety or environmental risk. Conversely, it may create an economic risk, but that is left to the operator's discretion. Clearly, such a philosophy can only be adopted where there are no doubts concerning effective de-manning.

The reduced metocean criteria/loads outlined in Section 9 of 2SIM are only intended for use in the fitness-for-purpose assessment of platforms designed and constructed to earlier editions of APR RP 2A WSD. 2SIM clearly states that they should not be used for:

- the design of new platforms
- the change of use of a platform
- the re-use of a platform.

The metocean criteria provided within 2SIM are to be applied with wave/wind/current force calculation procedures specified within API 2A WSD: 2014 (22nd edition). For platforms located in the GoM, Table 4.2 reproduces the metocean criteria to be utilised in applying what is referred to as a design level assessment. Clearly, these relate to return periods that are substantially smaller than those that would be used for a new design.

Category	Design Edition		
	API 2A-WSD 19th Edition & earlier	**API 2A-WSD 20th & 21st Edition**	**API 2A-WSD 22nd Edition & later**
L-1	50-year	100-year	100-year
S-2	Not applicable	Not applicable	Not applicable
C-2	15-year	50-year	50-year
L-3	10-year	25-year	25-year

Table 5.2: Design level metocean criteria for existing GoM structures based on Table 5, Section 9.4.1.2 2SIM.

In considering the S-2 platforms in the GoM, a design level assessment is not recommended. An assessment of these platforms should instead be performed using ultimate strength methods; the recommended metocean criteria based on the risk relating to life safety and the consequences of failure.

2SIM again highlights the risks associated with WID loading noting that:

> "*Platform damage and failure experience in the U.S. Gulf of Mexico clearly demonstrates that platforms are much more susceptible to damage if waves inundate the platform deck; however, calculation of wave forces on a deck and on the deck equipment is not a simple task*".

To overcome this issue 2SIM requires the use of the silhouette method; but states that other methods can be used provided they are justified.

In practice, the 2SIM recommended practice for the re-assessment of existing structures uses the same analysis methods as those outlined for current new designs but applies reduced metocean criteria provided these have no implications for life safety and environmental impact.

6.0 NORSOK requirements

The NORSOK standard N003:2016 (edition 3), hereafter referred to as N003, specifies general principles and guidelines for the determination of characteristics actions and action effects for the design, verification and assessment of all types of offshore structures used in the petroleum industry. Whilst it is primarily written for facilities on the Norwegian continental shelf, the principles outlined are more widely applicable. Importantly, the forward to the document specifically states that, where relevant, the NORSOK standards will be used to provide the Norwegian industrial input to the international standardisation process. Subject to the development and publication of international standards, the relevant NORSOK standard is withdrawn. Given that this standard has recently been updated, it should be treated as going beyond the present ISO standard; effectively ISO+. The present review will concentrate on the additional requirements.

In terms of accompanying documents, N003 provides the input to NORSOK N001; the latter being the principle standard for offshore structures. In effect, N001 provides the equivalent of ISO 19900 and refers heavily to it. For the reason noted above, N003 is the key document appropriate to the present review. In making comparisons to the ISO and the API documents, considered in Sections 3 and 4 respectively, N003 is less prescriptive in terms of design parameters when compared to API, but provides more detail concerning the preferred calculation procedures when compared to ISO. That said, all three standards recommend the calculation of sub-structure loads based upon a “load recipe” involving deterministic regular waves and a calibrated load model. The difference between the standards lies in the caveats that are applied to the calculation procedures. In this respect N003 raises more concerns and adds a lot more caveats. In doing so N003 draws heavily on DNV RP C205 which will be reviewed at the end of this Section.

In general terms N003 is consistent with ISO, defining a ULS corresponding to an annual exceedence probability of 10^{-2} and an ALS of 10^{-4}. However, in detail, it incorporates some significant differences. In overview, these concern:

(a) The estimation of characteristic actions, particularly the use of environmental contours for nonlinear problems.
(b) The increased use of model testing, particularly in respect of air-gap and WID loading issues.
(c) The determination of wave properties, both crest elevation and kinematics, with specific mention of nonlinear effects.
(d) The importance of wave slamming and other nonlinear loading effects.
(e) The implications of climate change.

Taking each of these points in turn, the differences included within N003 are discussed as follows.

6.1 The estimation of characteristic actions

In addressing this task N003 begins by stating that for fixed structures behaving quasi-statically, hydrodynamic actions relating to a particular exceedence probability can be estimated using a design wave method. Moreover, for structural members whose dimensions lie outside the diffraction regime, $D/\lambda<0.2$, a regular Stokes 5th-order wave can be used for ULS and ALS analysis. In this sense, N003 is consistent with ISO and API. However, whilst this may be acceptable for the prediction of sub-structure loads (depending on the wave nonlinearity) it will not be acceptable for WID loads. This is acknowledged in N003 in Section 11.6.3 (see below). Moreover, it states that any design wave method must be based upon the results defining the wave height and crest height from a long-term analysis. N003 suggests that this long-term analysis can be undertaken using two essentially different approaches.

(i) The all short-term conditions approach, also referred to as the initial distribution method or the all sea states approach when applied to wave conditions.
(ii) A storm event approach.

Whilst both methods have their respective advantages, if good quality data is available N003 states that (ii) is preferable. The over-riding advantages of this approach are its focus on the design weather conditions, the statistical independence of events considered and the fact that several metocean characteristics can be included in the analysis. The disadvantages are that results may be sensitive to the selected storm threshold and the occurrence of outliers; both effects needing to be carefully considered.

However, N003 notes that if the problem is of a very nonlinear nature, as will often be the case for ULS and ALS investigations, an extensive model test programme may be necessary to model the short-term variability of all the important metocean conditions. If the problem is of an "on-off" nature, such as the occurrence of WID loading and/or wave slamming, this will be necessary. In this case a simplified and approximate approach, based upon environmental (or metocean) contours, may be relevant. This is a simplified approach to (i) and is summarised as follows:

(a) Establish the environmental contours of the relevant metocean parameters (often H_s and T_p) for varying exceedence probabilities.
(b) Identify the worst case metocean conditions along the contour of interest for the variable under consideration.
(c) For this sea state determine the distribution of the 3-hour maximum value of this variable.
(d) Estimate the value of this variable for a given exceedence probability based upon the α-percentile of this distribution.

The advantage of this method lies in its simplicity and the fact that the environmental contours are often given in standard metocean reports. The disadvantage is that it is an approximate method; the value of the variable being subject to rather large uncertainties. The main difficulty lies in the choice of α. N003 suggests that for ULS calculations $\alpha=0.9$, while for ALS calculations α=0.95. These values are expected to be slightly conservative provided the coefficient of variation of the 3-hour maximum value of the variable does not exceed 0.20-0.25.

In undertaking this approach, N003 further suggests that to identify the worse-case metocean condition (point (b) above) four 3-hour seeds is sufficient, whereas to identify the distribution of the maxima twenty 3-hour seeds is recommended. Our own experience of undertaking this approach suggests that these recommended numbers may be rather small.

Interestingly, if this process were undertaken in respect of crest heights the estimated value, being based upon laboratory observations, would include both amplifications above 2^{nd}-order and the dissipative effects of wave breaking. This would address at least two of the concerns noted earlier. However, if this value were then used to drive a regular wave model a new set of errors would inevitably be introduced due to the fundamental inadequacy of the adopted wave theory.

6.2 Model testing

N003 is very specific on the benefits of physical model testing, indicating when such tests should be undertaken and the nature of the required data. For example, it notes in Section 11.1.8.1 that hydrodynamic model tests should be carried out to:

> *(a) Confirm that no important hydrodynamic action has been overlooked (for new types of facilities, metocean conditions, adjacent structure).*
>
> *(b) Support theoretical calculations when available analytical methods are susceptible to large uncertainties.*
>
> *(c) Verify theoretical methods on a general basis.*
>
> *(d) Determine the action/action effects for complex problems where numerical methods are insufficient*.

In addition, it notes that:

> "*If the problem under consideration is of a very non-linear nature, an extensive model test program may be necessary to model the short-term variability for all important metocean conditions*".

This hints at a very important point touched upon in Section 2 (see below). For example, in respect of wave slamming it specifically notes that slamming actions may occur in steep waves and that model tests shall be performed to determine the corresponding actions. Moreover, it notes in Section 11.6.2.2 that:

"*Slamming is of a highly nonlinear and stochastic nature, hence the slamming action effect with annual probability q may occur in sea states with higher probability of occurrence. In* *special cases model tests of slamming actions for a large number of sea states combined with a long-term analysis may be needed to estimate the characteristic ULS and ALS slamming action effects*".

This specifically addresses the point raised in Section 2 whereby, with the occurrence of wave breaking, it is not clear that either the ULS (10^{-2}) or the ALS (10^{-4}) loading event is associated with the sea state (or wave event) having a similar probability of occurrence. Having recognised this possibility in respect of slamming, N003 is suggesting that laboratory testing provides the best means of quantification. The present authors do not disagree with this approach, but the extent of the required model testing will be very significant.

In Section 11.6.3, N003 states that:

> "*Since air-gap and wave in deck analysis are subjected to significant uncertainties, analysis methods should be validated by high quality model tests with due consideration to the large inherent stochastic variability and uncertainty in predicting deck impact actions*".

Furthermore, in Section 11.6.3.2 it notes a number of nonlinear effects that are not included in the prediction of WID loads (see below) and concludes by stating:

> "*High quality model tests are recommended to validate the air-gap analysis*"

Finally, in respect of green water effects Section 11.6.4 states:

> "*Determining the amount of green water entering onto deck, and computing flow velocities and resulting action effects on deck structures is more complicated. Analyses using advanced nonlinear methods and/or model tests are recommended for design purposes or for detailed assessment of existing structures*".

6.3 Effects beyond second-order and wave breaking

In considering air-gap and WID loading issues, N003 states that a minimum requirement is a positive air-gap above the wave crest with an annual probability of exceedance of 10^{-2}. This includes the area amplification and fully nonlinear effects (see below). However, it also states:

"*Due to the complexity and uncertainty associated with determining actions associated with waves hitting the platform decks; designing for a positive air-gap above the wave crest with an annual probability of exceedance of 10^{-4} is recommended*".

More specifically, N003 notes that where diffraction effects can be ignored (so-called transparent bottom fixed structures) the air-gap shall be determined based upon second-order (**or higher**) wave crest height and the inclusion of the difference between point statistics and area statistics. If detailed evaluation is not performed, a factor of 1.1 on the second-order crest height, for both ULS and ALS conditions, should be used to account for both higher order wave effects and the correction due to the use of area statistics. Whilst this number may not be large enough to accommodate both effects, it is more realistic than the 15% (maximum) recommended by ISO and API to accommodate area effects and, most importantly, it represents a first attempt to accommodate effects beyond second-order.

N003 also states that a wave event used to model an extreme WID load can either be an irregular wave or a regular wave. However, if the latter is used it:

> "must be shown to be conservative".

Furthermore, it is clearly stated that the wave event approach must include an assessment of the effects of breaking and near-breaking waves. This is re-enforced by an earlier discussion of wave kinematics (Section 6.2.1.2) in which a clear distinction is drawn between action effects that are sensitive and not sensitive to either the local crest elevation or the kinematics close to the crest. In those cases that are sensitive:

> "*Special consideration should be made for crest kinematics in sea states with near breaking waves or breaking waves*".

6.4 Wave slamming and other nonlinear loading events

In contrast to ISO 19902 which argues that wave slamming is only of local interest, and API RP 2A WSD which suggests it can be neglected, N003 places an emphasis on its effective determination. Indeed, it notes that it should be investigated with due consideration to:

(a) The effective rate of change of momentum.
(b) Viscous actions.
(c) Buoyancy actions.

For horizontal members in the splash zone, slamming coefficients of 3.0 are recommended for smooth cylinders and 6.0 for flat plates. These follow the recommendations of DNV-RP-C-205. In respect of WID loading it argues that the total load corresponds to the sum of inertia, slamming, drag and buoyancy. It notes that the simplified model of WID loading included in ISO 19902 and draws attention to the alternative method outlined by Kaplin et al (1995).

In respect of large volume structures, but not so large that linear diffraction dominates, N003 notes that local run-up (vertical jet-flows) can cause local vertical impacts at the column-deck intersections. This is also noted in ISO 19903. Furthermore, N003 concludes that while the principle effect of slamming and upwelling is local, possible global effects should also be assessed.

N003 also introduces a specific section on slamming caused by wave breaking. It first notes (correctly) that:

> "*Breaking waves occur in deep water as well as in shallow water. Spilling breakers are most common, but plunging breakers may also occur*".

This goes well beyond either ISO or API and is entirely consistent with the findings of recent model tests undertaken within the *CREST*, *SHORTCREST*, and LOWISH JIP's (see Latheef and Swan (2013), Karmpadakis et al (2019) and Karmpadakis & Swan (2020)). Furthermore, N003 notes that:

> "*The horizontal particle velocity under such waves may exceed the phase velocity by at least 20 %*", following DNV RP C 205.

This suggests that horizontal velocities predicted using regular wave theories (either a stream function solution or a Stokes 5th-order solution) will be grossly inadequate. N003 states that vertical surfaces can be exposed to actions from breaking or near-breaking waves and that this should be checked with respect to both the ULS and ALS conditions. Moreover, it notes that numerical predictions of these actions will be associated with large uncertainties and that methods used for numerical predictions of slamming actions should be validated by high quality model tests.

In respect of nonlinear wave actions (Sections 6.3.5.2) N003 notes that wave impact/slamming from steep or breaking waves may cause a dynamically amplified resonant response. In such cases the nonlinear effect may be dominant, and where the results cannot be checked against previous experience they should be checked against model tests. In a Norwegian context, the motivation for some of this concern lies in the unexpected 'ringing' excitation of large (concrete) gravity-based structures. However, N003 is clear that dynamically amplified resonant responses may also be initiated by WID loads arising due to steep or breaking waves acting on fixed steel jacket structures.

6.5 Climate change

N003 directly raises the issue of climate change noting that the relevant global models predict an increase of 6-8% in extreme significant wave heights in the eastern North Sea through the 21st century, a rise of 8% in extreme wind speed over the same period, and a corresponding increase of 0.4-0.7m in mean sea level. Whilst these effects are perhaps not that significant on their own, extending well beyond the expected life of most structures, they are indicative of a clear trend; one that hints at the growing importance of the threat posed by WID loading.

6.6 DNV RP C205

This is a document prepared by DNV (now DNVGL) concerning recommended practice (guidance) in respect of environmental conditions and environmental loads appertaining to a wide range of offshore structures. The object is to provide rational design criteria and guidance for the assessment of loads on marine structures subject to wind, waves and currents. In effect, the document compiles a wide range of research results that are deemed relevant and practical to incorporate in engineering design. This document is heavily referenced by NORSOK N003:2016 and is clearly the source of some of the significant updates included within the NORSOK standard.

The present summary does not seek to review the entire DNV document, but concentrates on those parts of the recommended practice that are specifically noted in N003 and believed to be important in terms of the present review. These points are discussed as follows:

6.6.1 Wave kinematics

N003 notes that if an action effect is sensitive to kinematics close to the wave crests a second-order random process shall be adopted for the surface process. It then quotes DNV RP C205 (hereafter referred to as C205) for the modelling of the consistent second-order process and corresponding kinematics. In this regard it is important to note that the suggested kinematics are not necessarily second-order, merely 'consistent' with the second order surface. Indeed, DNV suggests three possible alternatives:

(i) Grue's method
(ii) Wheeler method
(iii) A second-order kinematics model

In considering these options, the present authors consider (iii) to be the preferred option, but do not consider this to be sufficient when the wave is very steep.

6.6.2 Wave breaking

N003 is the only standard to include the possibility of breaking waves in all water depths. Specifically, it notes:

(a) In respect of wave theories:

"*Special consideration should be made for crest kinematics in sea states with near breaking waves or breaking waves*". In this regard "*The horizontal particle velocity under such waves may exceed the phase velocity by at least 20 %*"

(b) In respect of wave slamming:

"*Vertical surfaces can be exposed to actions from breaking or near-breaking waves. The structure shall be checked against these actions during the design phase both with respect to ULS and ALS*".

(c) In respect of WID analysis:

"*A design wave approach should include the following: ..... the effect of near breaking and breaking waves shall be assessed*".

(d) In respect of nonlinear actions:

"*Actions from wave impact/slamming from steep or breaking waves on GBS structures or waves impacting jacket topside may cause a dynamically amplified resonant response*."

These comments are, in part, motivated by Section 8.8 in C205. This addresses the occurrence of breaking wave impacts noting that:

- The impact velocity should be taken as 1.2 times the phase velocity.
- The most probable largest breaking wave height may be taken as 1.4 times the most probable largest significant wave height in *n*-years.

Whilst the present authors agree that the first point provides an approximate upper bound, the second point is less certain and critically dependent on the associated wave period. As such, it is difficult to predict a hard upper-bound to the breaking (or non-breaking) wave height.

Nevertheless, the comments incorporated in C205 are very important since they introduce a different and potentially more threating category of incident waves that cannot easily be modelled by commonly applied wave models, particularly those based upon regular waves.

6.6.3 High-frequency action effects

In this regard, N003 notes that:

"*nonlinearities in the waves, and nonlinear interactions between waves and structure, may introduce important higher order action effects that may excite the structure at frequencies both below and above the frequency range of the linear wave components*".

Furthermore:

"*High frequency wave action effects are relevant for restrained modes and local dynamic action effects*".

In explaining this latter effect it adds:

"*For slender vertical columns where drag-induced wave action is dominant, a resonant response may be excited by second and higher order wave actions from integrating the horizontal drag forces up to the actual instantaneous free surface level*".

This emphasises the need to describe the fully nonlinear wave profile.

The motivation behind these comments is driven by DNV’s recommended practice in respect of “springing” and “ringing”, but have been applied more generally in terms of nonlinear dynamic response within N003.

7.0 Concluding remarks

Following the completion of this review there are clearly areas in which the standards fall well short of the present state-of-the-art in terms of scientific understanding. These are listed as follows and raised as possible areas for future code development:

(1) An assessment of recent changes in the short-term distribution of crest heights, including the competing influences of nonlinear amplifications beyond second-order and the dissipative effects of wave breaking; both assessed for realistic values of directional spreading, with a rigorous quantification of area effects.

(2) The accurate prediction of the wave-induced water particle kinematics throughout the water column; particular emphasis being paid to the largest, highly nonlinear, velocities arising close to the instantaneous crest elevation.

(3) The combination of waves and currents, particularly the reduction in the current velocities high in the wave crest due to the divergence of the streamlines.

(4) The effectiveness of a calibrated sub-structure load model (or recipe) involving regular waves.

(5) The over-emphasis/reliance on failure modes at (or just below) the sea bed involving:

  (i) The total base shear

  (ii) The total over-turning moment

  Whilst it may be argued that these are simply adopted as indicative measures of the applied loads, a substantial change in the vertical profile of the water particle kinematics and hence the applied loads (particularly with the occurrence of wave breaking) raises questions as to whether this can ever be a viable approach. This is particularly relevant to failure mechanisms that arise at high elevations within a structure.

(6) The accurate prediction of wave-in-deck loading, including a full time-history appropriate to its inclusion within a dynamic analysis.

(7) The abandonment of inaccurate and inappropriate calculation procedures simply because they are easy to use and have historical precedence, unless it can be shown that they are consistently conservative.

(8) The effective and accurate convolution of the short-term distribution of loads with the long-term distribution of sea states to produce the long-term distribution of loads. With the inclusion of nonlinear effects, including wave breaking, it is no longer sufficient to assume that the largest sea states produce the largest waves and these

define the largest loads. This issue needs to be addressed in the context of alternative failure modes (point (5) above).

(9) The inclusion of wave breaking, across the full range of water depths, with specific attention paid to:

- (i) Its influence on the crest height distribution.
- (ii) Modifications to the area effect.
- (iii) Changes in the predicted water particle kinematics
- (iv) The balance between sub-structure and wave-in-deck loading.
- (v) The implications for local loading and the onset of dynamic excitation.

(10) The occurrence and importance of wave slamming, particularly where it involves breaking waves.

Whilst these may eventually form the basis for a series of recommendations for the future development of the design codes, this cannot be confirmed until the relative importance of the effects has been established in the subsequent deliverables.

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