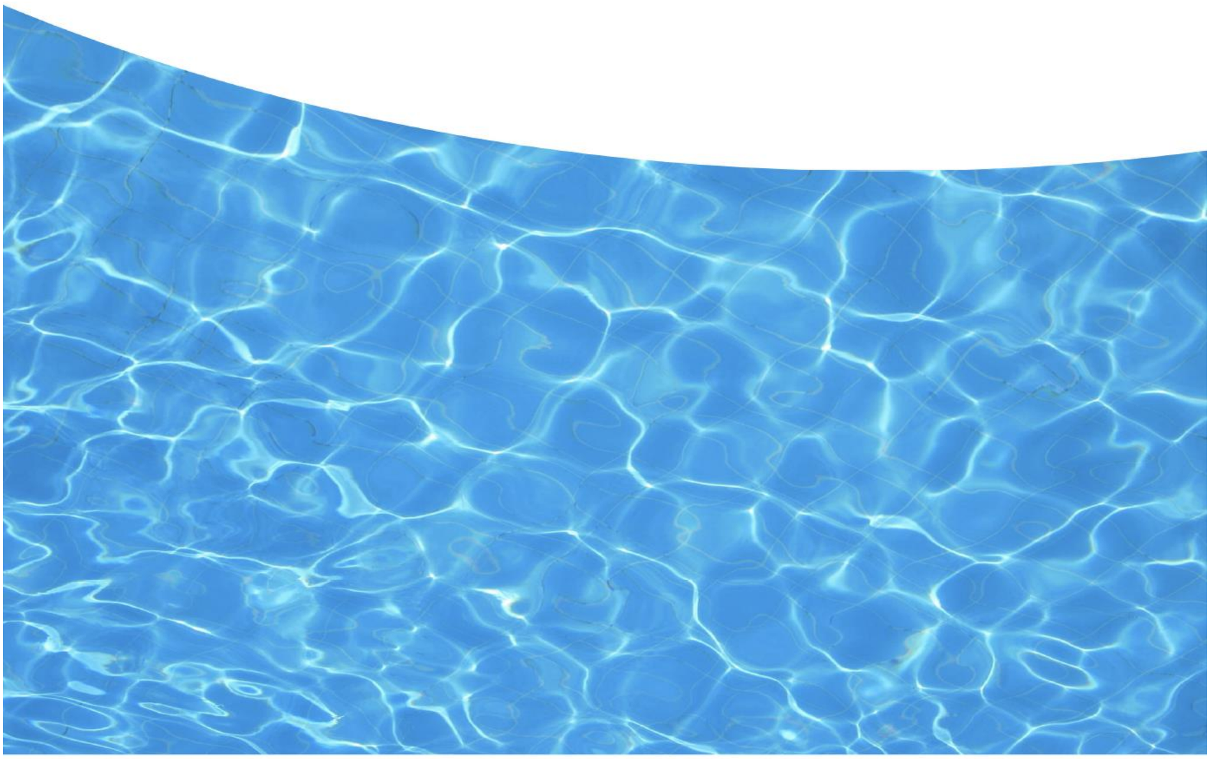
Extreme Environmental Loading of Fixed Offshore Structures: Summary report, Component 1

24th April 2020



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OFFSHORE CONSULTING GROUP

Revision History

Revision	Description
B	Conclusions added and references updated.
A	Original report.

Contents

1.0 Introduction

This report forms part of the research study into 10,000-year return period extreme environmental loading. The overall aim of the study was to develop guidance on the management of the risks to the structural integrity of fixed offshore structures exposed to extreme environmental loading. This comprises two complementary objectives:

Objective 1: Review of current prediction methods for, and the provision of recommendations on, the effect of extreme environmental loads on the structural integrity of fixed offshore installations.

Objective 2: Development of a risk-based framework for assessing the structural integrity of fixed offshore installations.

This report is the final deliverable associated with objective 1; deliverable O1/D8. It provides a written summary of the findings/conclusions arising from the first six deliverables which are listed as follows:

O1/D1:	Review of current code requirements for extreme environmental loading on fixed offshore structures.
O1/D2:	Review of the procurement and analysis of met-ocean data.
O1/D3:	Met-ocean research relevant to fixed offshore structures.
O1/D4:	10,000-year crest heights, wave kinematics and fluid loading.
O1/D5:	Directional met-ocean analysis.
O1/D6:	Extreme loading events and target reliability levels.

This report should be read in conjunction with the summary presentation prepared as deliverable 01/D7. This presentation includes many of the key results from which the conclusions included herein are drawn.

2.0 Context

As far as UK waters are concerned, the reliability of fixed offshore structures is principally concerned with the re-assessment of existing structures, rather than the development of design criteria for new structures. This is fundamentally important to the nature of the required reliability calculations. When considering the wave loads acting on an offshore structure, one of the largest loading components, potentially the most damaging, and certainly the most difficult to describe, is the wave-in-deck (WID) load. A new build structure is not designed to withstand such loads, but rather to avoid them. This is achieved by setting a deck elevation such that the maximum predicted wave crest elevation, over the plan area of the structure, remains beneath the deck. Such calculations must take due account of any potential sea bed settlement and the long-term trend in the predicted extreme crest heights. Unfortunately, the deck elevations relevant to many existing structures do not satisfy these requirements. As a result, a rigorous assessment of WID loading needs to be included within any reliability study.

The re-assessment of an existing structure is often associated with life extension which inevitably involves ageing infrastructure. In such cases the level of deck inundation, appropriate to the calculation of WID loads, may be substantial. The explanation for this lies in both:

- Changes in the target limit state design conditions
- Improvements in both the short-term and the long-term representation of the ocean environment.

Specifically, the re-assessment of existing structures in UK water will be subject to:

(a) Enhanced safety requirements associated with the imposition of a target 10^{-4} annual probability of failure. This may be very different to the 10^{-2} conditions on which many existing structures were originally designed.

(b) Increases in the predicted long-term sea state severity, characterised by increases in the significant wave height, H_s.

(c) Possible increases in the short-term distribution of crest heights, η_c arising in a given sea state; the latter characterised by H_s, T_p, σ_ϑ and $S_{\eta\eta}(\omega)$

(d) Improved representations of the water particle kinematics and hence the applied loads given the best possible representation of the incident wave, including the occurrence of wave breaking.

(e) Possible submergence of the sea bed due to de-pressurisation of the underlying reservoir.

In undertaking a reliability study, the goal is to identify the return period of the environmental loading that first causes global failure; the applied loads exceeding the structural resistance. In making this assessment, the primary cause of uncertainty lies on the definition of the applied loads. It is in this area that the most significant recent advances have been made and it is these changes that now need to be incorporated into present best practice for design/ re-assessment. This document seeks to summarise the required changes.

In most circumstances the total environmental loads are dominated by two components; the wave-in-jacket (WIJ) loads and the wave-in-deck (WID) loads. Both are proportional to the square of the fluid velocity; the WIJ loads calculated using Morison's equation and the WID loads a variety of local and global methods based upon drag, momentum, or some combination of the two. Although both WIJ and WID loads are important, the WID loading is the most difficult to define, exhibits the greatest variability and is associated with the largest uncertainty. The explanation for this lies in two parts:

(i) The WID load is often described as "badly-behaving"; a very large increase in the load arising due to a relatively small change in the level of inundation

$$\Delta\eta = \eta_{c,max} - h_d, \quad [2.1]$$

where $\eta_{c,max}$ is the maximum incident crest height and h_d the deck elevation. This is, in part, dependent upon the nonlinearity of the largest incident waves, but also on the characteristics of the topside structure.

(ii) The WID loading is only dependent upon the properties of the largest wave crests, close to the instantaneous water surface. This is the most difficult part of any wave form to describe, is subject to the fastest evolution in both space and time, and will exhibit the largest departures from the simplified steady (or regular) wave solutions on which traditional design solutions have been based. Furthermore, if wave breaking occurs, as will be the case in many of the largest waves (Latheef & Swan, 2013 and Karmpadakis et al, 2019), it occurs in exactly that part of the wave that causes the WID loading.

In practice, the occurrence of the WID loading and its effective prediction is the single most important factor determining the reliability of many fixed offshore structures.

3.0 International standards and design guidance

In completing deliverable 01/D1, particular attention was paid to the *API*, *ISO* and *NORSOK* standards. Having completed the reviews it was concluded that these standards do not represent the present state-of-the-art in terms of solution procedures that are currently available. Indeed, the fact that many North Sea operators acknowledge this, adopting solutions that go well beyond existing standards, provides clear evidence that recommended practice, particularly in respect of WID loading, needs significant updating.

Areas of particular concern relate to the use of:

- Input parameters based upon the long-term distribution of sea states, the prediction of associated parameters and the effective inclusion of uncertainty.
- Second-order crest heights
- 'Equivalent' regular waves, which are both steady and non-breaking
- Calibrated load models (recipes), where the extent of the calibration does not cover the full range of relevant wave conditions (particularly the occurrence of wave breaking).
- An over reliance on failure modes occurring at the sea bed: total base shear (BS) and total overturning moment (OTM).

In listing these points it is important to stress that they will not all necessarily be important to all studies. Nevertheless, they have the potential to produce non-conservative results, do not provide the best representation of the underlying physics, and must be addressed.

In considering the present standards, it should also be noted that *NORSOK* raises many more caveats and questions; seeking confirmation that the recommended calculation procedures are conservative. In addressing many of the issues noted above, this is achieved through a requirement for extensive model tests. In this sense *NORSOK* goes well beyond either *API* or *ISO*. For example, it makes specific mention of crest heights beyond second-order and notes the potential importance of wave breaking. In this regard it suggests the maximum fluid velocities (high in the wave crest) could be defined by $u_{max} > 1.2c$, where c is the phase velocity. Moreover, it suggests that the effects of wave breaking should be investigated experimentally. These effects can be important for WIJ loading and are critical for WID loading. As such, they should be addressed in all codes.

4.0 Data inputs

This primarily relates to the procurement and analysis of met-ocean data. The relevant met-ocean criteria, particularly the long-term distribution of extremes, can vary significantly depending on the era in which they were derived, the data on which they were based, and the methods of analysis. Traditionally, the required criteria were either based on field measurements or hindcast models. Unfortunately, field measurements are often of short duration and plagued by instrument biases; recent research having cast doubts over the accuracy of wind, wave and (particularly) current measurements. Alternatively, hindcast models such as NEXTRA or NORAID are limited to a few decades in duration. As such, estimates of the 100-year or 10,000-year criteria clearly require extrapolation. Moreover, hindcast models are calibrated using recorded field data. This is usually achieved using a variety of regression-based methods, but seldom address the short-term variability due to the sample duration (Forristall et al, 1996).

More recently, synthetic models and the application of climate model wind fields to drive very long simulations of wave conditions (as used in the NS1200 study) lead to a database that is at least two orders of magnitude longer than the hindcast models. This means that the 100-year event is interpolated from the data, rather than extrapolated, and the uncertainty in the 10,000-year estimate is reduced by approximately half.

5.0 Long-term distributions

Historically, these were based upon a Weibull distribution fit to all sea states within a hindcast model using least squares to derive omni-directional criteria. There are several limitations to this approach, most of which invalidate the extreme value assumption of independent, identically distributed data. These are outlined as follows:

- Consecutive sea states are not independent and therefore extreme value analysis methods cannot be easily applied.
- The Weibull is a distribution for minima not maxima. Whilst it can provide stable estimates of extremes, it is not a theoretically valid distribution for this problem; there is no guarantee that it will converge.
- The least squares fitting technique is dependent on the plotting position (such as Weibull or Gringorten) employed. This can produce important differences to large return period estimates.
- Omni-directional criteria do not consider directional effects such as different fetch lengths, sheltering, storm tracks or sand banks in shallow waters. Direction is a key co-variate that should be considered in design, as discussed by Jonathan and Ewans (2007).
- A single threshold is typically used to fit the Weibull distribution. This is a largely subjective choice resulting in a 'good' fit based on visual inspection and experience. Nowadays, there are methods of extreme value analysis, such as Bayesian model averaging over a range of thresholds, that are based on statistical rigour and remove the subjectivity.

Given these difficulties, the recommended approach is similar to that proposed by Ross et al (2017). The key features to be included are listed as follows:

(i) Independent events are identified using a peaks-over-threshold approach.
(ii) A Generalised Pareto distribution is fitted to the extremes and includes suitable covariates.
(iii) Joint distributions are modelled using Heffernan & Tawn (2004) and includes suitable covariates.
(iv) The temporal variation in Met-ocean parameters over the course of a storm should be modelled.
(v) Threshold uncertainty is included in the analysis.
(vi) The model parameters are estimated using Bayesian inference.

In adopting this approach, the uncertainty in extremes should be considered, and included in the analysis, unless this has already been accounted for in the calibration of the safety factors or design recipe.

5.1 Associated parameters

Historically, the 100-year wave height, 100-year wind speed and 100-year current speed were assumed to act simultaneously, all based on individual extrapolations. This led to conservative associated criteria that significantly over-estimated the 100-year response of the structure and did not lead to an optimised design. More recently, the most common approach involves individual regression of each associated parameter to the significant wave height, H_s, during storm peak events within a hindcast model or measured data. This is simple to apply and independent of the structural form. However, it is only applicable when one met-ocean parameter dominates the environmental loading. It does not consider the joint occurrence of multiple met-ocean parameters and only considers the behaviour of observed data rather than extremes.

As an alternative a more rigorous response-based analysis approach developed by Tromans & Vanderschuren (1995) is applied. This bases the criteria on the 100-year response of the structure rather than the independent 100-year met-ocean conditions. This incorporates the joint occurrence of many met-ocean parameters in the observed dataset and considers the relative directions between winds, waves and currents. However, it is dependent on the structural form, assumes the shape of the short-term distribution and the number of storms, and considers them invariant when extrapolating from the observed data to the extremes.

More specifically, individual parameters are discussed as follows:

- The wave period associated with the maximum individual wave height, T_{Hmax}, is typically based on a fraction of T_p that is rarely justified in met-ocean criteria reports. Sometimes it is taken from field measurements, but these are more benign than the design conditions and when numerical simulations are used to model the extreme sea states, they typically do not incorporate nonlinear effects.
- Associated currents are often ignored as they are difficult to incorporate unless a (simplified) load model is employed.
- There are many methods for incorporating the associated still water level with various degrees of accuracy. Most of these are based on simple regressions or empirical relationships and do not consider the statistical dependency between H_s, tide and storm surge.

Given these difficulties, it is recommended that:

(a) The associated conditions be derived using a single long-term model for the met-ocean environment. This ensures that different extremes are mutually consistent.

(b) Associated wave periods and currents should be determined using a simple structural load model. This should include the full irregularity, nonlinearity and directionality of the wave field, including the effects of wave breaking.

(c) Associated still water elevations should be determined using a full joint model for the environment in which the temporal profiles of waves, surge and tide through a storm event are captured.

5.2 Directional Met-ocean analysis

In most locations the characteristics and severity of the met-ocean environment varies with direction. It therefore follows that this should be included in any analysis of an offshore structure to avoid unnecessary over-conservatism. In considering the long-term assessment of met-ocean conditions, three key points arise:

- From an historical perspective an omni-directional analysis has commonly been applied. However, this is known to result in bias and inaccurate estimates of the extremes.
- The simplest method of incorporating directionality is to split the data into discrete directional sectors. The drawback with this approach is that it assumes that the met-ocean environment is piecewise constant. In reality, this is unlikely to be the case.
- A more realistic approach is to allow the parameters to vary in a continuous, possibly smooth, manner. Two approaches are applied: a Fourier series representation (Jonathan & Ewans, 2007) and splines based on a piecewise polynomial basis function (Randell et al., 2016); the latter typically providing improved stability and robustness of fit.

Having chosen how to represent the directional variation, the next step involves the development of directional criteria. Comparisons between the various methods lead to the following points:

- Omni-directional criteria are never non-conservative, but often very conservative.
- A traditional scaling approach in which the most onerous sector is set equal to the omni-directional value is rarely conservative, and often very non-conservative.
- The method suggested in Forristall et al. (2004) seeks to provide directional criteria at return periods of *N*x*R* (where *N* is the number of directional sectors and *R* the return period). This does not necessarily lead to an accurate assessment; some sectors having criteria that are larger than the omni-directional value. This approach is never non-conservative, but often very conservative.
- The NORSOK recommended approach lies somewhere in-between the traditional scaling approach and the Forristall et al. (2004) method. It suggests calculating the *N*/2x*R* return period value for each sector, but limiting values such that none is larger than the omni-directional value. This approach is often non-conservative, but rarely by very much.
- A slightly more conservative modification to the NORSOK approach is to calculate the *N*x*R* return period value for each sector and limit the values such that none is larger than the omni-directional value. This modified NORSOK approach is usually conservative, but rarely by very much.

The recommendations arising from this aspect of the study are three-fold:

(1) The long-term extreme value analysis of met-ocean data should include all relevant covariates. This will typically include direction.

(2) The best representation of directionality is achieved using splines as these allow a smooth, continuous, variation in met-ocean conditions.

(3) Directional met-ocean criteria should be developed using a modification of the NORSOK recommended approach, as described above.

6.0 Short-term distributions

Until recently, the short-term distribution of crest heights, or the distribution of crest heights given the sea state characteristics (H_s, T_p, σ_θ, $S_{\eta\eta}(\omega)$), was based upon the Foristall (2000) model. This corresponds to a two parameter Weibull fit to second-order random wave calculations based upon Sharma & Dean (1981); the fit expressed in terms of the sea state steepness and the Ursell parameter. This model provides a significant improvement over the linear Rayleigh distribution and is shown to be in good agreement with the crest height distributions measured in weakly nonlinear sea states. However, the sea states relevant to WID loading calculations, particularly where they relate to a 10^{-2} or 10^{-4} annual exceedance probability are not weakly nonlinear. Moreover, the individual waves associated with the occurrence of WID events, or those lying in the tail of the distribution, will be very steep (potentially breaking) and therefore highly nonlinear.

The first evidence of the nonlinear amplification of crest heights beyond second-order arose from the *SHORTCREST* JIP. With recent contributions, the evidence to support this view is now conclusive and summarised as follows:

(i) Comparisons between field data recorded in deep water and comparable laboratory testing (Latheef & Swan, 2013).

(ii) Confirmation of the laboratory testing using independent data recorded in two very different deep-water wave basins (Latheef & Swan, 2013).

(iii) An extension of the results into intermediate water depths including comparisons between available North Sea data and laboratory measurements (Karmpadakis et al, 2019).

(iv) Comparisons between crest height distributions recorded in intermediate water depths in three independent wave basins (Karmpadakis et al. 2019).

(v) Direct numerical calculations based upon a higher-order spectral model with appropriate dissipation to address wave breaking (Xiao et al, 2013) and Hadjigeorgiou, 2018).

In realistic sea states, the increase in the crest elevations above the Forristall (2000) model is dependent upon the competing influence of nonlinear amplification and wave breaking. In practice, recent calculations suggest that the nonlinear increase in design crest heights in deep water will be of the order of 5-8% (Latheef & Swan, 2013). Whilst this may not seem large, if it is this 5-8% that enters the deck, it can account for a substantial increase in the applied load and must be incorporated within any reliability analysis. In contrast, in shallower water locations, the increase in crest heights may be substantially less, perhaps lower than Forristall (2000) predictions. This arises due to the increased occurrence of wave breaking; particularly depth-limited wave breaking.

The recommendations of this study are that the short-term distributions of wave height and crest height are best predicted as follows:

(A) In deep and intermediate water depths, defined such that $k_p d > 1.2$

(i) Wave heights. The Forristall (1978) empirical distribution performs well for broad-banded sea states (γ=1) that are widely spread, since this corresponds closely to the hurricane conditions to which the distribution was fit. However, it is less well suited to large storms in the North Sea that are generally more narrow-banded and less directionally spread. In this case, the Boccotti (1989) distribution that incorporates spectral bandwidth effects is more appropriate.

(ii) Crest heights. In deep water these can be based upon the *SHORTCREST* JIP model. This model has been further refined in the *LOADS* JIP and its range of applicability extended to intermediate depths, subject to the limits noted above.

(B) In intermediate and shallower water depths $k_p d < 1.2$

(i) Wave heights. There are a number of available distributions that incorporate, to varying degrees, the relevant physics appropriate to shoaling and shallow water breaking. These include von Vledder (1991), Battjes & Groenendijk (2000) and Mendez et al, (2004). Each of them performs well in a given set of conditions but cannot be applied across the whole range of water depths (karmpadakis et al 2020a). Very recently, a new wave height distribution was developed within in the *LOWISH III* JIP. This compares very favourably with both field and experimental measurements across a wide range of intermediate and shallow water depths (Karmpadakis et al, 2017 & 2020b).

(ii) Crest heights. Whilst the Forristall (2000) model is very widely applied, comparisons with field measurements demonstrate that it can be very conservative. This arises because the Sharma & Dean (1981) model provides no upper-bound for wave breaking. In contrast, the crest height model developed in the *LOWISH III* JIP (Karmpadakis et al, 2017), building on the wave height model noted above, has no such limitations and agrees very well with available field data.

In respect of the crest heights, it is important to note that the recommended distributions relate to point statistics. If a structure encompasses a significant topside area, as is usually the case, the occurrence of WID loads must be based upon a rigorous assessment of the area maximum crest elevation. This is already included in the relevant standards and must be rigorously assessed. Forristall (2006) provides an appropriate model, but recent laboratory observations (Ma, 2018) suggest that this will be non-conservative in very steep sea states. However, with the occurrence of wave breaking, the area amplification is expected to reduce. Unfortunately, the present state-of-the-art is such that this change cannot be predicted without recourse to laboratory data.

7.0 Wave-in-jacket (WIJ) loading

Present practice for the calculation of WIJ loading is based upon a load recipe. This adopts a steady or regular wave theory, typically a Stokes 5th-order or high-order stream function, matched to target wave height based upon the required return period, H_{max}. The water particle kinematics, $\underline{\boldsymbol{u}}(x,z,t)$, predicted by this model are then adopted within Morison's equation and the WIJ loads predicted. In applying this method it is widely acknowledged that neither the water surface elevation, $\eta(t)$, nor the underlying kinematics, $\underline{\boldsymbol{u}}(x,z,t)$, provide an accurate representation of the actual design wave event; the latter being unsteady or irregular and directionally spread. However, the loading coefficients describing the drag, c_d, and inertia, c_I, loading components, are 'calibrated' to give the best possible description of the applied loads. The difficulties associated with this method are listed as follows:

(i) To achieve an effective calibration, appropriate to a range of wave steepness, effective water depths and structural forms, the adopted kinematics must exhibit a similar form to the actual kinematics. This is particularly relevant for $u_x(z)$, both immediately beneath the largest crest elevation and at all spatial locations within the plan area of the jacket. In practice, this is seldom true, particularly if the wave exhibits some level of breaking (see below).

(ii) Given significant differences in $u_x(z)$, the calibration will be dependent upon both the assumed failure mode and the elevation of failure. Traditionally, failure is usually expressed in terms of the total over-turning moment (OTM), or the total base shear (BS); both assessed at the mudline. Whilst this may be appropriate for some structures, it will not be appropriate for all.

(iii) The occurrence of wave breaking is known to be important for small exceedance probabilities, even in deep-water locations (Latheef & Swan, 2013). As a result, the magnitude of the near-surface velocities may be substantially increased and the vertical structure of the horizontal velocities, $u_x(z)$, markedly different. Such changes would require a different calibration (for non-breaking and breaking waves) and may provoke a different failure mode/elevation.

The consequence of these effects is that the bias associated with the application of a regular wave model is not consistent across different wave conditions, failure modes and structural configurations. That said, a design approach using regular waves could be calibrated for a carefully defined set of conditions. However, the development of multiple calibrations to accommodate all conditions is practically unrealistic.

To overcome this difficulty, whilst preserving the use of a deterministic design wave, the adopted wave model needs to be more realistic in terms of the included physics. In this way, the predicted kinematics will more closely resemble the actual, and hence the adopted calibration more widely applicable. To achieve this, a second-order random wave model is an obvious choice. The bias is reasonably small and is fairly consistent across most wave conditions, failure modes and structural configurations.

However, there is one key effect that it misses: the effect of wave breaking. Therefore, such a model is not appropriate for conditions or failure modes where wave breaking is important. An obvious example is the local loading of elements in the crest of a wave. In some instances, it is possible that this 'local' loading may lead to a global failure, or the failure of safety critical equipment. Whilst deterministic nonlinear irregular waves can provide a good approximation to the short-term distribution of loading in random seas, they do not (by definition) include the full stochastic variability within a sea-state. Therefore, in steep seas with many breaking events the distribution of loading can be under-estimated. However, it is possible to define a set of deterministic irregular wave events that encompass a range of wave steepness to account for this variability within a deterministic approach.

Building upon these comments, the recommended approach for a deterministic assessment of the WIJ loading associated with extreme wave events is to model a set of focused wave events that encompass a range of wave steepness. If the waves are not breaking, then their kinematics can be modelled using second-order theory. However, if they are breaking then a fully nonlinear model, or kinematics developed on the basis of laboratory testing, is required. The waves should be focused at a number of locations around a structure to ensure that a range of load cases and failure modes is considered. The properties of these waves must be determined on the basis of the long-term description of the met-ocean environment and this must include the relevant uncertainties.

As an alternative to the deterministic focused wave events outlined above, a full probabilistic analysis could be undertaken. This would normally be recommended when calculations involve very steep seas, with lots of wave breaking, and the possibility of WID loading. In such cases it becomes increasingly difficult to identify the small set of focused waves that represent all possible failure modes.

8.0 Wave-in-deck (WID) loading

The accurate and efficient prediction of WID loads is the most difficult part of any load calculation and (potentially) has the largest effect on the predicted reliability. The explanation for this is given as follows:

(i) The time-history of the WID load, $F_x(t)$, is dependent upon the incident wave shape, $\eta(t)$, the square of the associated fluid velocities, $\underline{\boldsymbol{u}}^2$, and the size/layout of the topside structure. Taken together, $\eta(t)$ and $\underline{\boldsymbol{u}}^2$ define the transfer of wave momentum into the topside, whilst its layout (particularly the openness or porosity of the structure) defines the rate at which this momentum is destroyed and hence the applied load time-history[1].

(ii) The WID loads are based entirely on that part of the wave crest that enters the topside structure, both through the front face of the structure (perpendicular to the direction of wave propagation) and through its underside. As a result, the only part of the wave that needs to be modelled lies close to a large wave crest for which $\eta_c > h_d$, where h_d is the deck elevation. Unfortunately, this is the most difficult part of any wave to model. It encompasses the largest fluid velocities, with the largest uncertainties, and is that part of the wave for which the "errors" in the kinematics predictions based upon a simplified design wave solution (using an equivalent steady or regular wave model) will be largest.

(iii) In Section 7.0, the difficulties associated with the development of an effective calibration, as part of a load recipe based upon an equivalent steady or regular wave, were discussed in respect of WIJ loading. Similar arguments apply in respect of WID loading. However, given the part of the wave profile that is of interest, point (ii) above, these difficulties become more acute. This is further complicated by the occurrence of wave breaking.

(iv) The nature of the WID loading is such that it involves a highly transitory (rapidly varying) free surface flow. In such cases it is unlikely to be well modelled by a classical drag force. That said, the applied loads will be proportional to $\underline{\boldsymbol{u}}^2$, with appropriate spatial averaging, as indicated in point (i). It therefore follows that if the WID loads are modelled using a Morison's type drag component, the empirical loading coefficients may lie well outside the expected bounds. Moreover, with the physics of the loading process inappropriately modelled, the loading coefficients may be subject to unexpected and rapid change. This significantly complicates the process of achieving an effective calibration, adding to the difficulties noted in (iii).

(v) Given the practical importance of wave breaking in all water depths, particularly in respect of the water particle kinematics, it cannot be assumed that the largest crest elevations producing the largest deck inundations will also produce the largest loads. If a smaller but steeper wave is breaking, the near-surface water particle kinematics will be larger giving a larger global load. As a consequence, if a structure is subject to a large level of deck inundation, the identification of the worse-case WID loading event, including the sea state in which it arises, becomes very problematic. In this case, a full probabilistic analysis may be required in which all sea states capable of producing a WID loading event are included within an assessment of the structural reliability. The first example of such an approach is outlined by Tychsen et al (2016), with a simplified approach developed within the *LOADS* JIP.

[1] This argument is based solely on the application of Newton's second law. If correctly formulated it will accurately reflect the physics of the loading process, allowing the WID loads to be predicted (See Ma & Swan, 2020b).

A full explanation of many of these effects, with reference to the available laboratory data, is given in a sequence of papers by Ma & Swan (2020a,b,c & d).

Given these difficulties, it is clear that the WID load models commonly used in design / re-assessment do not provide an adequate description of the applied loads. Specifically:

(a) The Graff et al. (1995) momentum flux model neglects the porosity of the topside structure and therefore produces a conservative estimate of the WID loads when based upon realistic descriptions of the wave shape, $\eta(x,y,t)$ and the water particle kinematics, $\underline{\boldsymbol{u}}(x,y,z,t)$.

(b) The silhouette method (recommended in the *API*, *ISO* and *NORSOK* standards, and recently updated by Santale, 2017) is typically non-conservative. This arises because of the calibration difficulties noted above and, specifically, the fact that the most commonly adopted calibration is based upon comparisons to the WID loads recorded by Finnigan & Petrauskas (1997) in which breaking waves were neglected. The present state-of-the-art suggests that there is no justification for this (Latheef & Swan, 2013 and Karmpadakis et al, 2019).

(c) The component model proposed by Kaplan et al (1995) incorporates a very broad range of force components and, as such, has the potential to be highly accurate. However, the extent of the required calibration (given all the difficulties noted above) is excessive, limiting the practical application of the model.

To over-come these problems, a new momentum flux model has recently been proposed (Ma & Swan, 2020b). This model incorporates the porosity of the topside structure, avoids entirely the need for empirical calibration, and has been very extensively validated in respect of both regular and random waves; the latter including the effects of wave breaking.

Taking due account of the issues noted above, the analysis of WID loads appropriate to reliability calculations can be undertaken in two ways:

(i) A full probability analysis in which all waves capable of producing WID loading events are modelled in all sea states arising in the long-term. This allows a full long-term description of the applied loads (both WIJ and WID) and hence a direct calculation of the structural reliability. Although a number of important short-cuts can be applied (full details of which are given in the *LOADS* JIP), this represents an extensive analysis. Nevertheless, in some cases (particularly those involving potentially large levels of deck inundation) such an analysis will be unavoidable.

(ii) A much simplified and deterministic analysis of the WID loads arising in one or more extreme wave events; the latter chosen to reflect the long-term distribution of the met-ocean conditions. In adopting this approach, it is essential that the calculated loads define an upper-bound to those given in (i), taking due account of the variability in the applied loads and all possible failure modes.

Importantly, the calculations undertaken in (i) or (ii) must incorporate the possible effects of wave breaking. In seeking to keep the calculations as simple/accessible as possible, the deterministic calculations outlined in (ii) will inevitably involve some degree of conservatism, hence the mention of an upper-bound. The challenge is to ensure that the extent of this conservatism does not become excessive. To achieve this the recommended procedures for the calculation of WID loads are described as follows:

(A) For structures experiencing a small level of deck inundation

(i) If the long-term met-ocean conditions suggest that the sea state steepness, S_1, and the effective water depth, k_pd, are such that the largest waves capable of entering the topside structure are non-breaking, the input wave conditions can be based upon a deterministic regular wave. This should be matched to η_{max} and T_{local}; the former including nonlinear amplifications beyond second-order and area effects, while the latter is based upon a focused wave event (for $k_pd \geq 1.0$). The predicted kinematics could then be input into a fully calibrated load model; the calibrations developed as part of a problem-specific recipe including the size and relative porosity of the topside structure.

(ii) As an alternative to the regular wave adopted in A(i), a second-order focused wave could be adopted. This would have two notable advantages. First, it would involve a wave case that is compatible with the recommended WIJ analysis (Section 7.0). Second, with a very significant improvement in the physics under-pinning the chosen wave event, it could either be applied within a calibrated load model (in which case the calibrations will be more widely applicable) or within the proposed momentum flux model (Ma & Swan, 2020b). In this latter case the need for an effective calibration is avoided. Indeed, no empirical input is required. Given that a focused wave evolves in both space and time, as would be the case for any large irregular wave, the position of the maximum crest elevation needs to be chosen such that it causes the worst-case load, talking due account of all relevant failure modes and elevations. Recent research suggests that this position usually relates to the arrival of the largest wave crest on the front face of the structure. However, this will inevitably be platform specific, depending on the layout of the topside structure. Furthermore, this would need to be considered in conjunction with the WIJ loads (Section 7.0); the phasing of the load components (WIJ and WID) arising above the failure elevation being critical.

(iii) If S_1 and k_pd (from the long-term met-ocean conditions) are such that the WID loading may be associated with large breaking waves, the calibration of a regular wave model becomes unrealistic. In this case a deterministic focused wave (as outlined in A(ii)) becomes more relevant. In modelling this wave, the crest elevation should be held constant, consistent with the target exceedance probability, and the local wave period reduced to create an over-turning wave event. This will define the required upper-bound, but should not be excessively conservative given that the level of inundation, $\Delta\eta$, is assumed small. Having defined the relevant wave event, the loads can be predicted in one of two ways:

(a) With numerically calculated wave kinematics used to provide the input to an effective WID load model (Ma & Swan, 2020b).

(b) The target waves generated experimentally and the WID loads measured directly.

In either case, the position of the focused wave crest relative to the topside structure must again be investigated so that the "worse case" loading event is identified, taking due account of all failure modes/elevations. As noted above, this should be investigated in conjunction with the WIJ loads, unless failure occurs immediately beneath the topside structure. In this latter case the WIJ loads become irrelevant.

(B) For structures experiencing a large level of deck inundation.

In this case a wide range of wave events can enter the deck structure, some of which will be breaking. Given earlier comments concerning the need to ensure that any deterministic wave event is conservative, it is important to note that the magnitude of any WID loads increases with:

- The maximum crest elevation, η_{max}, and hence the level of deck inundation, $\Delta\eta$.
- Reducing local wave period, T_{local}. For a given $\eta_{c,max}$, this produces a steeper wave with larger near-surface velocities.
- The severity of wave breaking; the largest loads associated with over-turning waves.

Acknowledging each of these points, a conservative estimate of the worst case WID loads will be produced by a deterministic focused wave matched to the underlying frequency spectrum, $S_{\eta\eta}(\omega)$, directional spread, σ_{ϑ}, and N-year crest elevation, $\eta_{c,max}$, in which the local period has been reduced to create an over-turning wave. In generating this case, the incident wave conditions could either be generated numerically (via a fully nonlinear wave model or CFD) or experimentally in a laboratory wave basin. Likewise, the load time-history could be produced using a validated WID load model (Ma and Swan, 2020b), direct from CFD using a pressure integration (feasible but computationally expensive and unproven for realistic porous topsides), or from laboratory observations. In each case, a variety of focused locations must be considered to identify the worse-case loading event for all relevant failure modes/elevations.

In adopting this approach, the method specifically sets out to maintain a level of conservatism. This is a necessary requirement of any simplistic deterministic calculation. However, in seeking this, the solution may prove to be excessively conservative. This is of particular concern for structures that experience a large level of deck inundation; the latter arising due to significant sea bed settlement or large variations in the long term met-ocean criteria. In such cases, the wave conditions associated with failure can occur over an increasingly large part of the H_s-T_p parameter space. The only way to avoid this level of conservatism is to adopt a full probabilistic analysis as outlined in (i) above.

9.0 Concluding remarks

Taken together, the deliverables produced within Objective 1 of this study confirm that the present state-of-the-art in terms of scientific understanding has moved well beyond the present code requirements. This conclusion is appropriate to:

- The description of the ocean environment
- The physics of extreme wave events
- The nature of the applied loads
- The calculation of the maximum loads
- The assessment of structural reliabilities / failure probabilities

Whilst not all of these effects will be relevant to all structures, they should be addressed in any re-assessment of structural integrity. Indeed, this is increasingly acknowledged within the offshore industry, with many owners/operators applying calculation procedures that go beyond present recommended practice. Indeed, in some cases these analysis procedures are consistent with the present state-of-the-art and have been responsible for driving it forward.

The challenge that remains is to update the existing codes to reflect these advances and, in so doing, ensure that all design / re-assessment achieves the target reliabilities.

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