



Wave-in-deck loads: an assessment of present design practice given recent improvements in the description of extreme waves and the nature of the applied loads

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ABSTRACT

This paper contributes to the on-going discussion of how best to calculate the reliability of a fixed offshore structure. This discussion has been driven, in large part, by improved physical understanding of waves arising in realistic design sea-states, with a growing appreciation that many 'design wave events' will be breaking, irrespective of water depth. As such, it is increasingly acknowledged that some aspects of present design practice are non-conservative. In re-assessing older structures, the accurate calculation of horizontal wave-in-deck (WID) loads is often the most important and least tractable part. This paper explains the underlying reason, highlights the wider implications for identifying an appropriate design point, and raises fundamental questions in the assumptions underpinning present practice. Specifically, a large laboratory data base of WID events is used to assess the success of available models. These comparisons confirm that recommended practice, including recent updates, consistently under-predict the maximum WID loads on which reliability calculations should be based. In contrast the recently developed Lagrangian Momentum Absorption (LMA) model (Ma and Swan, 2020b), a simple but complete load model that combines fully-nonlinear wave inputs and the openness/porosity of a structure, provides highly accurate predictions. This is achieved without empirical coefficients/calibrations and therefore ideally suited to design/re-assessment applications.

1. Introduction

When seeking to quantify the reliability of a fixed offshore structure subject to extreme environmental loads, all possible failure modes must be addressed and a balance sought between the available structural resistance and the applied loads. In effect, calculations are undertaken to determine the loading event with the largest annual exceedence probability that just causes failure. If conditions relate to the most extreme Accidental Limit State (ALS), both the load and resistance factors are neglected (set to 1.0) and the concept of survivability accepts that whilst some local damage may occur, a catastrophic failure, particularly one resulting in total collapse, loss of life or widespread environmental damage, is avoided (NORSOK, 2017). Having identified this limiting condition, provided its exceedence probability is smaller than the specified (required) target, the structure is deemed to have passed this reliability condition. Alternatively, if the target reliability level is set to an annual exceedence probability of 10^{-n} , or one failure in 10^n years, and it can be shown that a structure survives the 'worst case' loading event corresponding to this return period, then it also achieves its target reliability. Although these two approaches appear

closely aligned, the difficulty (and differences) arise in the definition of what is the 'worst case'.

Interestingly, it is often assumed that the largest load is generated by the largest wave (ignoring any associated wind and current) and that this corresponds to the 'worst case' load (or the load most likely to cause failure). Whilst this jump is more difficult to justify, it allows the target reliability to be specified in terms of the n -year wave rather than the n -year load. Although this has significant advantages in terms of the simplicity of the calculations undertaken, it becomes further removed from the identification of the worst case loading event. Since it is the latter that drives failure and hence defines the appropriate reliability, non-conservative results may occur.

In undertaking calculations of this type it is important to acknowledge that the largest source of uncertainty lies in the description of the environmental loads, not the structural resistance. Although international standards and recommended practice (ISO, 2013; API, 2014; NORSOK, 2017; DNV-GL, 2019) provide guidance as to the calculation of the applied loads, this does not represent the present state-of-the-art either in terms of modelling the most extreme waves or predicting the resulting loads. Indeed, given recent advances in the description

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of extreme waves and the spatial distribution of the resulting loads, there are aspects of present design practice that are at odds with the requirements of a reliability analysis. This arises because the latter must be based upon a rigorous assessment of the ‘worst case’ loads, taking due account of all possible incident waves.

The present paper continues in Section 2 with a discussion of extreme environmental loading, contrasting the differences between wave-in-jacket (WIJ) and wave-in-deck (WID) loads. Section 3 considers the fundamental characteristics of the large wave events responsible for these loads, describes how the WID loads are commonly predicted in present design practice, and questions the appropriateness of some of the methods employed. In Section 4 the success of the available methods is quantified by comparisons to a large data base of laboratory observations, describing the horizontal WID loads arising due to large wave events in extreme seas. The paper concludes in Section 5 with some practical recommendations concerning the effective modelling of WID loads and their incorporation within an assessment of structural reliability.

2. Extreme environmental loading

Addressing the case of a fixed, bottom-mounted, offshore structure, the applied wave loading will typically represent the largest contribution to the maximum environmental loads. This can be sub-divided into two parts: the sub-structure loads, hereafter referred to as the wave-in-jacket (WIJ) loads, and the wave-in-deck (WID) loads. With a typical jacket structure defined by a space frame fabricated from relatively small-diameter members, the WIJ loads will be distributed from the sea-bed up to the underside of the deck structure, dominated by drag loading (assuming the Keulegan–Carpenter number is large, >20) and calculated using Morison’s equation (Morison et al., 1950). This is a well-established approach in which the origin of the loads arises from flow separation and the formation of a downstream wake; the net force on an individual member arising due to the difference in the high pressures on the upstream face in the vicinity of the stagnation point and the low pressures in the downstream wake. Moreover, with an effectively calibrated loading ‘recipe’, both the base shear (BS) and the total over-turning moment (OTM) about the base can be accurately predicted. For historical reasons, primarily associated with accessibility and ease of use, such recipes have typically been based upon an ‘equivalent’ regular wave; the latter matched to the local wave period, T , and wave-height, H .

In contrast, the WID loading component is often treated as a point load, centred about that part of the deck which is inundated. Such loads only arise when the incident crest-elevation, $\eta_c(x, y, t)$, exceeds the elevation of the underside of the deck structure, $z = h_d$, at some point beneath the plan area of the deck. Within this notation, (x, y) are the horizontal co-ordinates, z is measured vertically upwards from the mean water level (MWL), and t is time. Such loads are only dependent upon that part of a large wave crest that enters the deck, highly transitory in nature, and often said to be ‘badly-behaving’. The explanation for this latter description lies in three parts:

- (i) The occurrence of WID loads produces a step-change in the magnitude of the total environmental loads.
- (ii) The load increases rapidly with the level of inundation, $\Delta\eta = \eta_c - h_d$.
- (iii) The WID loads exhibit large variability. This is due, in part, to spatial changes in the porosity of the topside structure, combined with the fact that within a random sea a large individual wave crest can enter any part of the deck structure, or all of it.

Further evidence of the difference between these two loading components is provided in Fig. 1. Sub-plot (A) concerns the total horizontal base shear (BS) acting on a typical jacket structure with a base area of $40\text{ m} \times 40\text{ m}$, in a water depth of $d = 150\text{ m}$, subject to a design sea-state defined by $H_s = 14.66\text{ m}$, $T_p = 16\text{ s}$ and a frequency-dependent Ewans

(1998) directional spread, where H_s is the significant wave-height and T_p the spectral peak period of a JONSWAP spectrum with a peak enhancement factor of $\gamma = 2.5$. Within Fig. 1(A), the WIJ contribution to the base shear, F_x^{WIJ} , is calculated using second-order random wave theory (Sharma and Dean, 1981) with realistic loading coefficients applied within a Morison’s equation formulation; F_x^{WIJ} expressed as a function of the incident wave-height, H , at the centre of the jacket; the latter determined from the time-history of the water surface elevation, $\eta(t)$. In Fig. 1(B) the corresponding WID loads acting on a topside structure with plan dimensions of $60\text{ m} \times 60\text{ m}$ are presented. The deck elevation was set at $h_d = 12.5\text{ m}$ for all loading events arising in this sea-state. These data were recorded in a laboratory wave basin, the short-term sea-state characteristics exactly as described above. In this case, the distribution of loads F_x^{WID} have been expressed in terms of incident wave crest-elevation at the centre of jacket, $\eta_{c,c}$; the fact that non-zero values of F_x^{WID} occur for $\eta_{c,c} < h_d$ reflecting that $\eta_{c,c} \neq \eta_{c,\text{max}}$ over the plan area of the deck structure.

Comparing the data presented in Figs. 1(A) and 1(B) confirms that the WID loads exhibit a much larger variability; the coefficient of variation (COV) of the WIJ loads defined by 0.35 and of the WID loads by 2.35. These should be taken as nothing more than example values; the COV for the WIJ loads being close to those reported by Heideman and Weaver (1992) and Efthymiou et al. (1997) in respect of field measurements at the Tern platform. Part of the explanation for this difference lies in the fact that the WIJ load is an integrated quantity, summed over the entire volume of the jacket structure. In contrast, the WID loads are defined by a highly localised loading event, always occurring at the very highest elevations in the largest wave crests. The nature of the free-surface boundary conditions is such that these locations exhibit the largest non-linearity and hence the greatest variability. Nevertheless, this variability should be predicted and included in a rigorous assessment of the structure’s reliability.

When seeking to describe the largest waves in a realistic design sea-state that is broad-banded in both frequency and direction, the relevant events will be unsteady, directionally spread, highly non-linear and, potentially, breaking. Until recently, such waves could not be accurately described. Indeed, the wave models necessary to achieve this (Bateman et al., 2001, 2003; Hague and Swan, 2009; West et al., 1987; Dommermuth and Yue, 1987) are still not in common usage. For entirely understandable reasons, not least relating to accessibility and ease of use, international standards are expressed in terms of simplified regular wave models used in conjunction with a loading recipe. In adopting this approach there is an acknowledgement that the predicted wave shape, and the associated water particle kinematics, will be inaccurate, but that the differences can be calibrated out through the use of empirically modified loading coefficients. The justification for this approach is that it is the load predictions that describe the occurrence of failure and hence the reliability.

Furthermore, from a historical perspective, it was assumed that:

- (i) The provision of an effective air-gap eliminated any occurrence of WID loads, and
- (ii) The WIJ loads could be represented by integrated quantities describing either the BS or the OTM, both assessed at the mud-line.

Given these assumptions, it is easy to understand how the poor kinematics predictions associated with a regular wave solution, combined with an empirically calibrated load recipe, can give satisfactory load predictions. However, this only applies if the calculated loads are not critically dependent upon the largest, most uncertain, and least-well predicted water particle kinematics arising high in the wave crest, close to the instantaneous water surface. Indeed, if these kinematics become important (perhaps even dominant) then it is difficult to see how a loading recipe could be used with confidence, without updated and problem-specific calibrations. Having identified this issue, two situations arise in which important questions need to be asked.

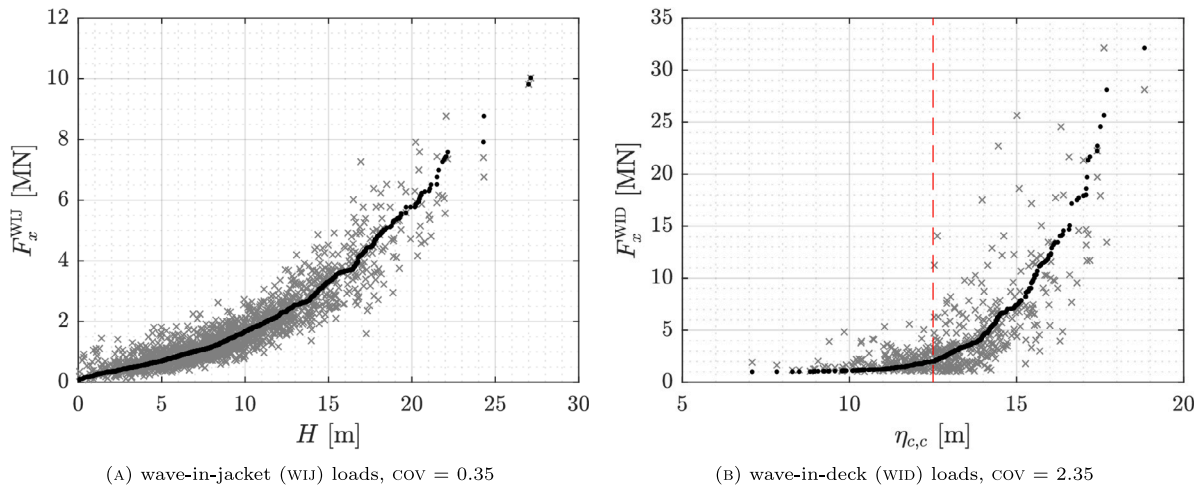


Fig. 1. Two distinct load components contributing to the total base shear; example data showing the variations with the incident waves arising in a sea-state defined by $H_s = 14.66$ m, $T_p = 16$ s, a JONSWAP $S_{\eta}(\omega)$ with $\gamma = 2.5$ and σ_p defined by [Ewans \(1998\)](#) in a water depth of $d = 150$ m. (A) $F_x^{WIJ}(H)$, to be consistent with present design practice and (B) $F_x^{WID}(\eta_{c,c})$ for a $60\text{ m} \times 60\text{ m}$ topside structure with an average porosity of $P = 63\%$. In each sub-plot: [x] wave events plotted against their corresponding loading magnitudes, [•] a quantile-quantile (Q-Q) ranked plot defining the most probable load for any given incident wave event (H or $\eta_{c,c}$), and [—] the elevation of the underside of the deck structure, h_d .

The first relates to the prediction of WIJ loads when the failure mode arises at increasing elevations above the sea-bed. In this case the only loads that matter are those occurring above the failure elevation since it is these that drive the failure mode. As the elevation of the failure increases, the loads become increasingly dependent upon the most uncertain velocities arising close to the instantaneous water surface. In this case a calibration based upon either the BS or the OTM at the mud-line becomes increasingly irrelevant.

The second question concerns the additional effects of WID loading. If this now becomes important (for the reasons noted below), can a simplified loading recipe be used for this part of the problem as is presently recommended?

3. Recent advances in the description of design events

Although it is an extreme loading event, rather than an extreme wave event that defines structural reliability, recent improvements in our understanding of both need to be incorporated in a rigorous assessment of present design practice. Given that the waves drive the loads, they will be addressed in that order.

3.1. Extreme waves

Having recognised that all the wave events of interest will arise in sea-states that are broad-banded in both frequency and direction, will exhibit significant non-linearity (due to their size/steepness) and may potentially be breaking (either spilling or over-turning), two immediate issues arise. First, recent research has shown that the short-term distribution of crest-heights (within a given storm) will be larger than the second-order predictions on which design calculations are typically based. In all but the shallowest water depths ($kd < 0.5$, where k is the wave number and d the water depth), crest-heights are presently predicted using the [Forristall \(2000\)](#) model; the latter based upon a fit to a large number of calculations using the second-order random wave theory of [Sharma and Dean \(1981\)](#). Evidence of an increase in crest-heights is provided by both [Latheef and Swan \(2013\)](#) and [Karmadakis et al. \(2019\)](#); the latter being particularly significant since it also provides agreement with extensive field observations. Although the increases in crest-height may not be large (typically 5%–10%), they can be very important in determining the occurrence of WID loading. Indeed, if it is this 5%–10% of the crest-height that enters the deck, then it can have a profound effect on the calculated reliabilities.

Second, there is increasing evidence of the importance of wave breaking. Whilst this has long been recognised in shallow-water coastal environments, its importance in intermediate and deeper water depths is only now becoming apparent. This is particularly relevant to the extreme, low-probability (or ‘tail’) of a crest-height distribution. Evidence of this was first discussed by [Latheef and Swan \(2013\)](#) in relation to a number of deep-water sea-states relevant to design conditions in the North Sea (and elsewhere). It was also raised by [Tychsen et al. \(2016\)](#) in relation to reliability calculations undertaken in the Danish sector of the North Sea. More recently, [Karmadakis and Swan \(2020\)](#) have considered the average shape of the largest waves arising in both laboratory and field data, demonstrated good agreement between them, and explicitly identified the occurrence of wave breaking; the latter providing a dissipative mechanism capable of reducing the crest-height associated with individual wave events, thereby shifting them to higher exceedence probabilities. Having undertaken a systematic laboratory investigation of the crest-height distributions arising in sea-states of increasing steepness, with all other spectral parameters held constant, direct comparisons to second-order predictions on a wave-by-wave basis allowed the separation of the wave field into un-broken and broken waves. This, in turn, enabled the percentage of breaking waves to be explored; particular attention being paid to the low-probability tail of the distribution. The conclusion drawn from this work is that many of the largest waves responsible for WID loading will also be subject to some degree of wave breaking. Whilst this does not imply that all such waves are over-turning, it does suggest that the properties of breaking waves must be incorporated if a reliability analysis is to be accurate. In this regard, the new short-term crest-height distribution proposed by [Karmadakis and Swan \(2022\)](#) which includes non-linearity beyond second-order and the effects of wave breaking represents an important step forward.

In considering representative wave shapes, [Karmadakis and Swan \(2020\)](#) considered the highest 5% of waves, separated the non-breaking and breaking events, normalised each profile by its maximum and calculated an average. In comparing these results, there are marked differences between the non-breaking and breaking waves; the latter exhibiting a greater crest-height asymmetry, a steeper front face and a narrower wave crest, particularly at the highest elevations relevant to WID loading. Importantly, there are also very significant differences between the shapes of the largest (and steepest) wave events arising in realistic random/irregular seas and those commonly adopted for engineering design. These differences arise in both time, $\eta(t)$, and

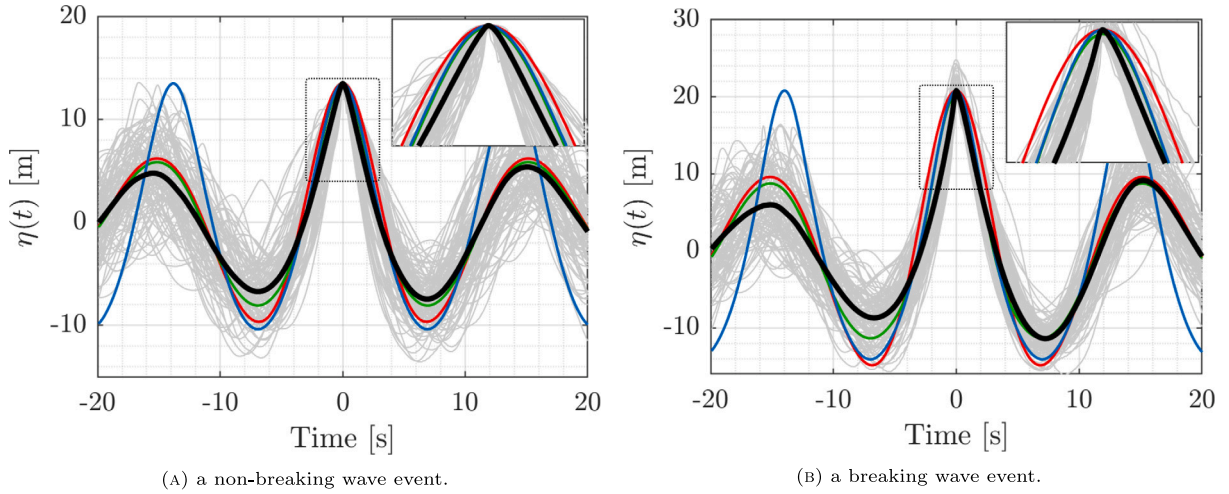


Fig. 2. Representative design wave events extracted from laboratory data involving long random simulations of realistic seas with comparisons to commonly adopted design wave solutions. (A) Large non-breaking waves in a sea-state defined by $H_s = 14.66$ m, $T_p = 16$ s, a JONSWAP $S_{\eta\eta}(\omega)$ with $\gamma = 2.5$ and σ_θ defined by Ewans (1998) and (B) the largest limiting/breaking waves in a steeper sea-state with $H_s = 17.2$ m and all other parameters unchanged. On each sub-plot: [—] individual large waves, [—] averaged wave profile and predictions based upon a [—] Stokes 5th-order solution (Fenton, 1985), [—] linear and [—] second-order QD focused wave, respectively.

space, $\eta(x)$, occur in both non-breaking and breaking waves (the differences being larger in the latter) and arise due to the fundamental physics that governs the evolution of the largest/steepest waves in realistic seas. Specifically, they relate to the unsteadiness (or spectral bandwidth), non-linearity (or steepness) and the directionality (or short-crestedness). Whilst these effects are each important in their own right, they are also strongly inter-dependent. For example, the non-linearity of the largest waves in a sea-state is critically dependent upon the spectral bandwidth and the directionality. Likewise, with increases in the non-linearity, there can be local changes in both the spectral bandwidth (Gibson and Swan, 2007) and the directional spread (Latheef et al., 2017). In addressing the underlying physics, the key parameters can be defined both in terms of sea-state averaged parameters and at the level of an individual or local wave event. These can be markedly different. Given that it is individual wave events that define the WID loads, the present paper will focus on the properties of the individual waves rather than the sea-state as a whole.

In terms of international standards and recommended practice (ISO, 2013; API, 2014; NORSOK, 2017; DNV-GL, 2019), present design procedures are either based upon a non-linear regular wave (calculated using a Stoke's 5th-order solution such as Fenton (1985), or a higher-order stream function solution following Dean (1965) or a linear/weakly non-linear quasi-deterministic (QD) focused wave based upon the theory of Lindgren (1970) or Bocchetti (1983) and popularised by Tromans et al. (1991). Evidence of the difference between the typical design wave events and the average shape of the largest waves arising in two realistic sea-states is given in Fig. 2; sub-plot (A) concerns the average shape of 100 non-breaking waves observed in the sea-state considered in Fig. 1; the sea-state steepness defined by $\frac{1}{2}H_s k_p = 0.117$, with all spectral parameters as noted previously. In contrast, sub-plot (B) describes the average shape of 100 larger/steeper waves which were at or very close to their breaking limit; the events observed in a steeper sea-state ($\frac{1}{2}H_s k_p = 0.137$) in which $H_s = 17.2$ m with all other spectral parameters held constant. Both examples are drawn from laboratory data based upon 60 three-hour simulations; the individual wave events being time-shifted to align the crests and averaged. This average, or representative large wave shape, is first compared to a Stoke's wave profile based upon the maximum crest-elevation, $\eta_{c,max}$, and the local trough-to-trough wave period, T_{local} . In addition, it is also compared to both a linear and a second-order simulation of a QD focused wave based upon $\eta_{c,max}$ and the underlying wave spectrum, $S_{\eta\eta}(\omega, \theta)$ where ω is the wave frequency and θ the direction of wave propagation.

In addition to the marked changes in the wave profiles observed in Fig. 2, particularly high in the wave crest, Fig. 3 considers the vertical profile of the maximum horizontal velocity in the mean wave direction, $u(z)$, beneath the two wave crests identified in Fig. 2. In both examples, comparisons are made between exact numerical calculations based upon a multiple-flux boundary element method (BEM) (Hague and Swan, 2009) and the typical design wave solutions; the latter again based upon a Stokes 5th-order wave theory, a linear QD focused wave with Wheeler (1969) stretching and a improved second-order QD focused wave. In all cases the solutions are matched to the averaged wave record as described above. These results suggest that the typical design solutions significantly under-estimate the maximum horizontal velocities arising high in the wave crest, particularly in the case of a breaking wave. Moreover, they also produce a poor description of the vertical profile, $u(z)$. In considering the data presented on Fig. 3, it is easy to see that if the calculations of the WIJ loads is based upon a full depth-integration ($-d \leq z \leq 0$), appropriate to failure at the mud-line, an empirical calibration can be introduced to ensure $\int u^2 dz$ based upon any of the design solutions are matched to the correct/target value based upon the fully non-linear BEM calculations. However, as the depth of integration reduces, corresponding to failure modes at elevations above the bed, the calibration factor ceases to be relevant, will become increasingly platform-specific (depending on member sizing and spacing) and must be re-evaluated for all possible failure elevations/modes. Most importantly none of these calibrations, particularly those based on failure at the mud-line, are relevant to the calculation of WID loading. This is only dependent upon the velocity of the fluid that enters the deck and must, by definition, incorporate the very largest velocities that occur at the highest elevations within the wave crest.

3.2. Wave-in-deck loading

Given the potential importance of WID loading, both in terms of its occurrence and its magnitude, the requirements of a successful WID load model can be summarised as follows:

- (1) It must be based upon a sound physical understanding of the applied loads, ensuring wide applicability.
- (2) Capable of producing a load time-history consistent with the prediction of dynamic excitation.
- (3) Appropriate to a broad range of incident wave conditions and topside geometries/layouts.

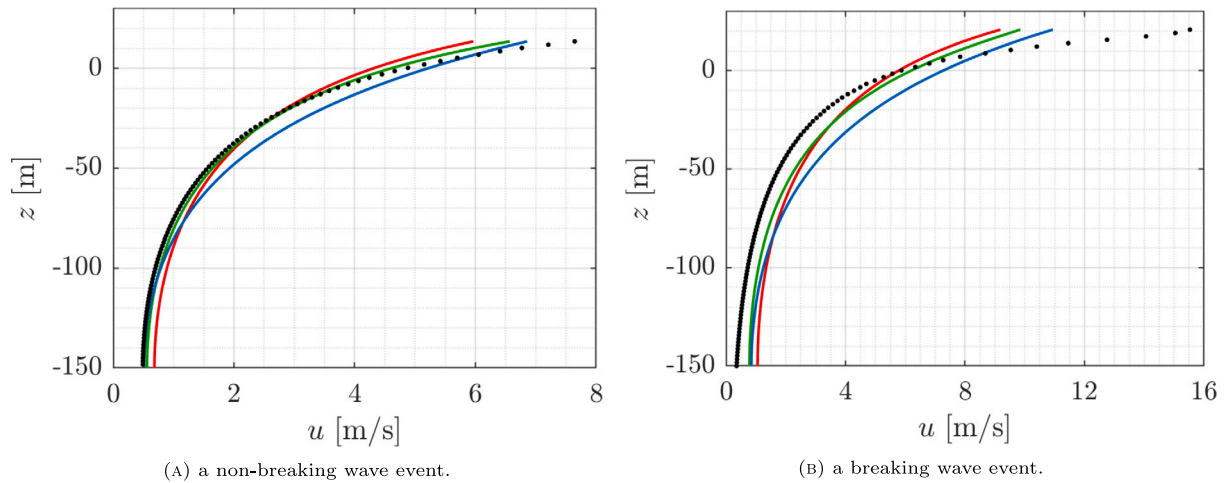


Fig. 3. Depth-variation in the horizontal water particle kinematics, $u(z)$, arising beneath the crests of the design wave events described in Fig. 2. On each sub-plot are predictions based upon a [•] fully non-linear BEM solution, [—] Stokes 5th-order solution (Fenton, 1985), [—] linear (with Wheeler stretching) and [—] second-order QD focused wave, respectively.

- (4) Employ minimal empirical inputs, preventing the need for problem-specific calibrations.
- (5) Simple and quick to apply, acknowledging the complexity of the required solution.

In seeking to characterise the available models, the underlying methodology can either be:

- (I) A global approach, or
- (II) A component approach.

In the first instance the total WID loads are based upon the gross properties of the topside structure such as plan area (A), inundation ($\Delta\eta$), and overall porosity ($\bar{P}$). In the second case the total loads are based upon the sum of the loads acting on individual components (or groups of components) defining the topside structure. Whilst the latter approach is undoubtedly more flexible, it usually requires substantially more empirical input.

A second characterisation could be based upon the included loading components. This will vary in complexity and could involve:

- (a) A model solely based upon a drag-type formulation.
- (b) A multiple loading component model including drag, inertia, slamming or impact and buoyancy.
- (c) An exchange of momentum from the wave crest to the topside structure; essentially a direct application of Newton's second law.

At first sight the loading components outlined in (a), (b) and (c) appear different. However, there are important similarities. For example, the drag term included within a Morison's-type formulation is proportional to u^2 . Likewise, a slamming or impact force and an exchange of momentum will also be proportional to u^2 . In such cases, the challenge that arise is to determine if and when a loading coefficient is required and how best it should be physically interpreted. As such, it would be instructive to take each of the commonly adopted WID load models in turn, characterise them in terms of the criteria noted above and assess the extent to which they meet the desired requirements listed in (1) to (5).

The simplest and most widely applied WID load model is the so-called *silhouette method*. This defines the maximum WID load in terms of a drag formulation, such that:

$$F^{\text{WID}} = \frac{1}{2} \rho C_d u^2 A, \quad (1)$$

where ρ is the density of the fluid, u the incident fluid velocity, C_d the drag coefficient and A the wetted area. This model can be characterised as an I-(a) formulation, is recommended in all the international

Table 1

Recommended loading coefficients, C_d , for the silhouette method based upon API (2014); the higher values (marked out by *) recommended by DNV-GL (2019).

Deck type	End-on and broadside drag coefficient C_d	Diagonal (45°) Drag Coefficient C_d
Heavily equipped	2.5*	1.9*
Moderately equipped	2.0	1.5
Bare (no equipment)	1.6	1.2

standards/guidance notes (including API, 2014, ISO, 2013, NORSOK, 2017 and DNV-GL, 2019), and is applied as part of a loading recipe. In this latter regard, u is calculated using a non-linear regular wave theory, matched to H_{max} and $T_{\text{associated}}$, and C_d empirically calibrated based upon a fit to laboratory data (Finnigan and Petruskas, 1997); the latter describing the WID loads acting on topside structures of varying porosity. Information concerning the recommended loading coefficient is given in Table 1; this includes the higher values recommended by DNV-GL (2019).

In terms of its physical justification, the highly transitory nature of a WID loading event ensures that the loads will not be drag-dominated in a practical sense. The explanation for this is that there is insufficient time for the development of a downstream wake (see earlier comments relating to the origin of the drag forcing). Indeed, as a wave passes through a topside structure, much of the loading will develop before there is any fluid (water) on the downstream side of many of the loaded components. Given this loss of physical reality, it becomes difficult to justify a drag coefficient (typically based on combined Reynolds number and Keulegan–Carpenter number effects) and to identify the range of flow parameters over which it is relevant. This is not to say that the model is invalid, simply that it becomes very difficult to justify without problem-specific calibration, covering the full range of incident wave conditions.

It is also important to note that the silhouette method does not immediately define the load time-history, $F(t)$. This can be remedied by adopting u and A as a function of time, or using an assumed (design) load time-history; an example of the latter proposed by van Raaij and Gudmestad (2007).

An alternative global model based upon the rate of transfer of momentum from the wave crest into the topside structure was proposed by Graaf et al. (1995). This is characterised as a I-(c) formulation, the load time-history defined by:

$$F_x(t) = \frac{\partial}{\partial t}(mu) = u \frac{\partial m}{\partial t} + m \frac{\partial u}{\partial t} \approx u \frac{\partial m}{\partial t} = u \cdot \rho u A \quad (2)$$

where m is the mass of fluid, $\mathbf{u}$ is the velocity vector (u, v, w) and the subscript x refers to the x -component. Within this equation the final approximation arises because $\frac{\partial u}{\partial t} \rightarrow 0$ high in the wave crest; hence the reference to quasi-stationary conditions in Graaf et al. (1995). Also, the final term $\rho \mathbf{u} \mathbf{A}$ is the mass flow rate of fluid entering the deck. This has two components ($\rho u A_x$ and $\rho w A_z$); the former entering through the front face and the latter the underside. For a simple rectangular box of width b , the resulting face-load, $F_f(t)$, and base-load, $F_b(t)$, are defined by:

$$F_f(t) = \rho \cdot u(x, t)^2 \cdot [\eta_f(t) - h_d] \cdot b \quad (3)$$

$$F_b(t) = \int_{x_c(t)}^{x_f(t)} \rho \cdot w(x, t) \cdot u(x, t) \cdot b \, dx \quad (4)$$

where $(\eta_f(t) - h_d)$ is the submergence on the front face, and the limits of the horizontal integral define the region $(x_f - x_c)$ over which the rectangular topside is both submerged ($\eta > h_d$) and ($w > 0$); both necessary to ensure the input of momentum into the topside structure.

In defining this model Graaf et al. (1995) assumed that all of the momentum arriving at the boundaries of the topside structure was instantaneously dissipated. This neglects the possibility that some momentum:

- (i) May be deflected before it reaches the topside structure.
- (ii) Will enter a topside structure and be progressively dissipated as it moves through the structure.
- (iii) Will exit an open structure having not been fully dissipated.

As a result, the Graaf et al. (1995) model is widely assumed to provide an upper-bound solution for WID loads.

In contrast, the Kaplan et al. (1995) WID load model is a detailed component model characterised as a II-(b) formulation. However, in respect of horizontal forcing, the model acknowledges that in many structures, but not all, the outer boundary of the deck incorporates relatively short vertical sections that are effectively solid. The forcing on these elements is defined by the sum of an inertial momentum change and drag such that:

$$F_x^{\text{WID}}(t) = \frac{\partial}{\partial t} (M_{a,x} \cdot u) + \text{drag} = M_{a,x} \frac{\partial u}{\partial t} + u \frac{\partial M_{a,x}}{\partial t} + C_d \frac{1}{2} \cdot \Delta \eta \cdot B u |u|, \quad (5)$$

where $M_{a,x} = \frac{2}{\pi} \rho \cdot \Delta \eta^2 \cdot B$ defines the added mass of a vertical plate of length B which is submerged to a depth $\Delta \eta$, assuming $\Delta \eta \ll B$. All other notation is as described previously. Eq. (5) was first given in Kaplan et al. (1995) and is repeated in DNV-GL (2019), the latter also noting that for a thin flat plate normal to the flow $C_d = 1.9$, provided the Reynolds number exceeds 10^4 . In addition, Kaplan et al. (1995) acknowledges that other types of elements, including vertical, horizontal, and inclined cylindrical sections will attract additional forces including inertial hydrodynamic forces, impact, drag and buoyancy. Indeed, it is argued that the inclusion of these forces is necessary to achieve good agreement with physical model test data, the analytical models adopted being based upon existing solutions, including classical Morison-type models. In adopting Eq. (5) together with a detailed component approach, two concerns become apparent:

- (a) Great care must be exercised to avoid the possibility of ‘double counting’ when calculating the summation of lots of different loading components. For example, if an incident flow causes an impact, it cannot subsequently drive a drag load. If it does, the latter will be substantially reduced in magnitude because of the initial impact. In essence, the application of any type of load must involve a consequent reduction in the momentum associated with the incident flow. This reduced momentum cannot, subsequently, drive a second or third loading component; any element of the incident momentum can only be destroyed or transferred once.

- (b) Earlier concerns, relating to the appropriateness of a drag force in a highly transient free surface flow close to the instantaneous water surface, are applicable to both terms in a classical Morison’s approach. In his seminal contribution, Lighthill (1979) discussed the physical origins of both the drag and inertia terms in Morison’s equation; the inertia driven by the integral of the potential flow pressures around a body, hence his reference to potential flow forcing, and the drag forcing driven by viscous flow separation, hence his reference to viscous or vortex flow forcing. In the case of WID loading it is unclear if either of these mechanisms is physically relevant.

Setting aside these issues, Kaplan et al. (1995) also argued that in the case of a partially open or porous screen (using the example of a grated deck) the first two terms in Eq. (5) would be multiplied by a correction factor $S^{\frac{3}{2}}$, where S is the ratio of solid area to the total projected area. This is closely related to the porosity P , adopted by Ma and Swan (2020b), such that $S = 1 - P$. The chosen power, $3/2$, converts an area ratio to a volume ratio so that it is dimensionally consistent with terms involving the added mass which naturally involve a volume. Likewise, Kaplan et al. (1995) also argued that the third term in Eq. (5), the drag force acting on area ($\Delta \eta \cdot B$) must be scaled by S to account for the actual wetted area.

Finally, Kaplan et al. (1995) also noted that where multiple elements are in close proximity, such as a riser/conductor array, they are treated as a group to account for velocity blockage effects. This accounts for the reduced total fluid loads acting on the group due to the incident flow being deflected around the body (Taylor, 1991). Moreover, the subsequent reductions in the flow, after the fluid passes a number of such groups as the wave crest propagates through the topside structure, is introduced using an empirical shielding parameter. In principle, the component approach proposed by Kaplan et al. (1995) has the potential to be highly accurate, but requires extensive and problem-specific calibration to define the loading coefficients, blockage factors and shielding parameters.

In an attempt to realise the benefits of a component approach, whilst removing the need for empirical calibration and maintaining the simplicity (and hence speed) of WID load calculations, Ma and Swan (2020b) proposed a Lagrangian Momentum Absorption (LMA) model. This is essentially a II-(c) formulation in which an incident wave crest is time-marched through a topside structure. At each time-step, that part of the wave crest about to enter the topside is sub-divided into a large number of momentum parcels (MPs). These are then individually tracked (in a Lagrangian fashion) through the topside, dissipating their momentum depending on the (structural) elements or groups of elements they encounter. Whilst the LMA model is entirely based upon the conservation of momentum, essentially a direct application of Newton’s second law, the implications for mass conservation also need to be addressed. At the beginning of the load calculations, when the MPs enter the topside structure, both the momentum and the fluid velocities are accurately defined. This implies that the appropriate mass must also be correctly defined when summed over all MPs. As the MPs move within the topside, no attempt is made to enforce the conservation of mass. However, the fact that the dissipation of momentum (based solely upon the porosity of the topside elements each MP encounters), together with the recommended method of velocity adjustment (the latter simply used to define the trajectory of the MP based upon its percentage loss of momentum) gives the correct load time-history suggests that the model is physically consistent when summed over all MPs. Indeed, the fact that the predicted load time-history is not critically dependent upon the velocity adjustments, clearly argued in Ma and Swan (2020b), confirms (as expected) that it is the total rate of dissipation of momentum that governs the success of the load model.

To maintain simplicity, the topside is typically represented by a combination of two-dimensional screens and three-dimensional sponges, where:

- A screen is typically used to model a blast-wall or line of conductors/risers; the momentum dissipation being proportional to $1 - P$.
- A sponge is often adopted to represent array of well-heads, manifolds, and pipework; the momentum dissipation being based upon $(1 - e^{-\frac{\ln P}{\Delta} \chi})$, where P is again the porosity (percentage of openness), Δ the average void size of the sponge and χ the distance an MP has travelled inside the sponge.

At each time-step the total rate of momentum change, summed over all MPs after encountering these elements, provides the global WID loading. Importantly, in each case the rate of dissipation of momentum is simply based upon the openness or porosity of each screen/sponge. This ensures that no empirical calibration is required. To achieve this, it should be noted that in applying the model the external boundary of the topside structure must be sub-divided into k rectangular panels and the time-step Δt pre-determined, as outlined in Ma and Swan (2020b). Whilst an inappropriate choice of k and Δt will inevitably affect the model accuracy, these parameters are both chosen to ensure a converged solution; k being increased and Δt reduced until the predicted load time-history becomes invariant. Full details of how the model is applied is given in Ma and Swan (2020b).

In a rigorous assessment of the accuracy of the LMA model, Ma and Swan (2020b) compared the model to the full range of laboratory data presented in Ma and Swan (2020a). This covers a very broad range of incident wave conditions including regular waves, irregular waves, varying levels of inundation, wave steepness, directional spreads and, perhaps most importantly, different types of wave breaking including limiting, spilling, and over-turning waves. Moreover, a follow-up study (Ma and Swan, 2023) also compared the model to a broad range of topside structures, including different porosities, orientations, and relative locations. In all cases the LMA model was shown to be in very good agreement with both available laboratory data and independent numerical calculations (Chen et al., 2018). More importantly, a detailed analysis of this data, including appropriate normalisation, confirms that the nature of the applied WID loading is fundamentally one of a momentum transfer from the wave crest to the topside structure. Indeed, this provided the motivation for the development of the LMA model and explains its broad success.

3.3. Design point

With the rate of momentum transfer dependent upon $u \cdot \rho u A$ (Eq. (2)), both the wave shape, $\eta(x, t)$, and the fluid velocity are important. For non-breaking waves, laboratory observations (Ma and Swan, 2020a) have shown that for a given level of inundation, the wave shape tends to dominate. This has important practical implications suggesting that calculations based upon a deterministic regular wave will, at worst, be conservative; an under-prediction of the fluid velocities being more than compensated by changes in the crest shape leading to an over-estimate of the volume of water entering the topside (Figs. 2(A) and 3(A)). Unfortunately, this will not always be the case. With the occurrence of wave breaking, the water particle velocities high in the wave crest will rapidly increase (Fig. 3(B)). In this case calculations based upon deterministic regular waves, which will never be steep enough to break ($\frac{1}{2} Hk < 0.44$), may become non-conservative.

This possibility is fully explored in Section 4. However, the difference between non-breaking and breaking waves raises a more fundamental concern for those structures with a relatively low topside elevation. To be clear, such occurrences do not imply a poorly designed structure. Indeed, it may be relevant to many older structures that were based upon less stringent design conditions (involving 10^{-2} rather than 10^{-4} exceedance probabilities) and/or outdated met-ocean criteria. Alternatively, the platform may have been subject to sea-bed subsidence following the depressurisation of the underlying reservoir. Whatever the cause, a relatively low topside elevation opens the possibility that

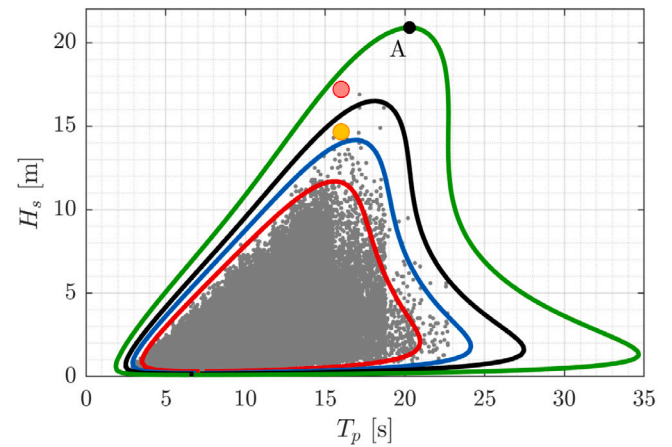


Fig. 4. Environmental contours describing the H_s - T_p variations for increasing return periods; the data representative of deep-water conditions in the northern North Sea: [•] individual sea-state from field observations and/or hind-cast data, and extrapolated contour lines corresponding to an annual exceedance probability of: [—] 10^0 (a return period of 1 year), [—] 10^{-1} , [—] 10^{-2} and [—] 10^{-4} . Sea-states from which the waves identified in Fig. 1 were extracted: [•] $H_s = 14.66$ m, $T_p = 16$ s and [•] $H_s = 17.2$ m, $T_p = 16$ s.

a broad range of wave events may enter the topside structure. In this case, it does not necessarily follow that the largest crest-elevations will produce the largest WID loads. Indeed, it may be the case that a smaller breaking wave produces the largest loads and hence the highest probability of failure.

The practical implications of this are significant; not least in terms of the range of sea-states that must be considered in any re-assessment of structural reliability. In most existing approaches, investigations are centred about the largest waves arising in the highest sea-states, or at the peak of the H_s - T_p environmental contour for a given n -year return period. For an exceedance probability of 10^{-4} this corresponds to point A on Fig. 4. Given that both the extent of any non-linear amplification and the occurrence of wave breaking is dependent upon the sea-state steepness (Lathief and Swan, 2013), best practice should explore sea-states of equal exceedance probability arising to the left of point A on Fig. 4. With a reduction in both H_s and T_p , and with sea-state steepness dependent upon $1/T_p^2$, these sea-states have the potential to be steeper and hence more likely to include breaking waves; the two coloured dots defining the sea-states considered earlier.

However, having acknowledged that smaller but breaking waves may produce increased WID loading, the relevant design sea-states could lie over a much broader region of the H_s - T_p contour space. Moreover, rather than the ‘design’ wave event occurring once in a 3-hour representation of an n -year sea-state, it may correspond to a much rarer event occurring in a more frequently occurring sea-state; the rarer event being a wave which achieves both a specified threshold in terms of its crest-elevation and is breaking. Whilst such events may not be relevant for structures with a high topside elevation, for which only the very largest waves can enter the deck, this may be very important for structures involving lower deck elevations. Evidence of this is provided by Tychsen et al. (2016) in their re-assessment of the reliability of jacket structures in the Tyra field in the Danish sector of the North Sea. In this case, reliability calculations based upon very long-duration random wave testing, covering an extremely broad range of sea-states, were markedly different from present design practice based upon deterministic wave events arising in sea-states at the peak of the H_s - T_p environmental contour.

4. Assessment of present design practice for WID loading

Within the sub-sections that follow the available WID load models will be compared to an existing database of laboratory observations

Table 2

The present (API) recommended application of the silhouette method (hereafter referred to as Method 1).

Step	Description
1.1	The calculation is based upon the n -year maximum wave-height, $H_{\max}$ and associated wave period, $T_{\text{associated}}$.
1.2	This is modelled using a non-linear regular wave solution, either Stokes 5th-order or Stream Function theory; the latter being more appropriate in shallow water.
1.3	The water depth, d , is adjusted (increased) to give the correct n -year crest-elevation.
1.4	The water particle kinematics are calculated based upon $H_{\max}$, T and d .
1.5	The silhouette method is then calculated using the correct inundation, $\Delta\eta$ and the empirically calibrated drag coefficients, C_d , based upon Finnigan (1991) and Finnigan and Petruskas (1997). In adopting these values, it is important to stress that they are used as part of a loading 'recipe'; the intention being that they should compensate for the acknowledged 'errors' in the kinematics predictions.

describing the WID loads produced by a very broad range of wave events acting on several different topside structures. In each case the models have been applied exactly as recommended in present practice or, in the case of the LMA model, as outlined in Ma and Swan (2020b).

4.1. Silhouette method

It has already been noted that the silhouette method (Eq. (1)) as outlined in ISO (2013), API (2014), NORSOK (2017) and DNV-GL (2019) is the simplest and most commonly applied WID load model. If the presently recommended procedure is described as Method 1, its application can be summarised in steps 1.1–1.5 outlined in Table 2. In considering this approach it has long been recognised that the n -year wave-height used in conjunction with a non-linear regular or steady wave model will describe an inaccurate crest-elevation, η_c , and hence an inaccurate inundation, $\Delta\eta = \eta_c - h_d$. Despite the critical importance of $\Delta\eta$ for any WID load predictions, this is the result of a clear (and in some respect understandable) desire to ensure that the WID loads are consistent with the WIJ loads; the latter traditionally based upon a deterministic regular wave defined in terms of an n -year $H_{\max}$. To overcome this difficulty, the water depth is adjusted as described in Step 1.3.

The fact that the crest-elevation associated with the n -year wave-height does not give an inundation, $\Delta\eta$, consistent with the n -year crest-elevation has been the subject of a recent API working group addressing the so-called 'API-RP-2MET crest conundrum'. The view of the present authors is that this is less of a 'conundrum', more a direct consequence of using an inappropriate regular or steady wave model. That said, the output from this working group, described in Santala (2017) defines a revised silhouette method, hereafter referred to as Method 2. This is to be incorporated in the latest revision of API-RP-2MET which is expected to be published soon. Details of this method are given in Table 3.

The essential difference between Method 1 and Method 2 lies in the iterative changes in d and T (steps 2.3–2.6); the purpose being to retain a description expressed in terms of the n -year $H_{\max}$, but at the same time maintaining both the correct inundation, $\Delta\eta$, and the correct $u_{\max}$ (at $z = \eta_{\max}$) based upon the n -year $\eta_{\max}$ and $T_{\text{associated}}$. This latter requirement arises from the original calibration of the model by Finnigan (1991) and Finnigan and Petruskas (1997), in which it was found that the adoption of this estimate of $u_{\max}$ led to a minimisation of $\Sigma(\Delta F_x)$, the sum of the differences between the measured and predicted WID loads.

In both Figs. 5 and 6 the two alternative applications of the silhouette method are compared to recent laboratory data presented in Ma

Table 3

The revised application of the silhouette method following Santala (2017) (hereafter referred to as Method 2).

Step	Description
2.1	As described in Method 1, Step 1.1.
2.2	As described in Method 1, Step 1.2.
2.3	As described in Method 1, Step 1.3.
2.4	Adjust T to achieve the correct $u_{\max}$ (at $z = \eta_{\max}$) given the n -year $\eta_{\max}$ and $T_{\text{associated}}$.
2.5	With a reduction in T , the wave steepness and hence the non-linearity increases. As a consequence, the crest-through asymmetry increases, leading to a larger $\eta_{\max}$ for a given $H_{\max}$. Having achieved an improved description of $u_{\max}$, there is now a mismatch between the target and predicted n -year $\eta_{\max}$ and hence the target and predicted $\Delta\eta$.
2.6	Iterate on 2.3, 2.4 and 2.5, incrementally adjusting d and T to achieve the target n -year $\eta_{\max}$ (and hence $\Delta\eta$) and the 'correct' $u_{\max}$.
2.7	Apply the silhouette method as described in Method 1, Step 1.5.

Table 4

Laboratory data base of deterministic WID test cases.

(A) incident waves.	
spectral peak period	$T_p = 1.4 \text{ s}, 1.6 \text{ s} \text{ and } 1.8 \text{ s}$
directional spread	$0^\circ \leq \sigma_\theta \leq 30^\circ$ and Ewans (1998) spreading
spectral bandwidth	$\gamma = 1, 2.5, 5$ and regular waves
inundation	$10 \text{ mm} \leq \Delta\eta \leq 50 \text{ mm}$
location of $\eta_{c,\max}$	$-300 \text{ mm} \leq x_f \leq 300 \text{ mm}$
(B) topside configurations.	
topside elevation	$95 \text{ mm} \leq h_d \leq 145 \text{ mm}$ from SWL
topside orientation	$45^\circ \leq \bar{\theta} \leq 90^\circ$ relative to the mean wave direction
overall topside porosity	plated, 30%, 40%, 63%
deck floor porosity	plated, 30%, 40%, 63%, no floor

and Swan (2020a) and Ma and Swan (2023); the former considering a very broad range of incident wave conditions acting on one specific topside structure and the latter a smaller subset of wave cases incident to a broad range of topside structures. Both experimental studies were conducted in the deep water wave basin within the Hydrodynamics Laboratory at Imperial College London. The incident wave shapes were first measured and fully defined using an array of 49 resistant-type wave gauges. These were then removed, and their space occupied by a generic topside structure; the latter being $0.6 \times 0.6 \text{ m}$ in plan dimensions, and modular to allow systematic variation in structural properties. The incident waves were then repeated; the WID loads measured at 12 kHz using a bespoke 6-degree-of-freedom load cell. Full details of these laboratory test cases are given in the earlier papers. For the purpose of the present discussion, it is sufficient to note that the incident wave cases considered herein were generated as quasi-deterministic (QD) focused waves, based upon the theory of Lindgren (1970), Boccotti (1983) and Tromans et al. (1991). With the linear inputs to the wave paddles defined according to this theory, the individual waves that evolve within the wave basin are fully non-linear and representative of the largest waves arising in long random wave simulations (Swan and Latheef, 2016). These cases are based upon realistic JONSWAP frequency spectra (Hasselmann et al., 1973) with varying γ and σ_θ . A summary of the incident wave cases and the topside configurations are given in Table 4. Overall, the study covered 86 incident wave events acting on 25 different topside configurations; the total number of deterministic loading events being well in excess of 1000. Importantly, the range of spectral peak periods and input amplitudes ensured that both non-breaking and breaking wave events were recorded; a subset of wave events also being generated at varying locations relative to the topside structure.

Fig. 5 concerns a subset of the loading events noted on Table 4; the wave cases all being focused on the front face of the structure.

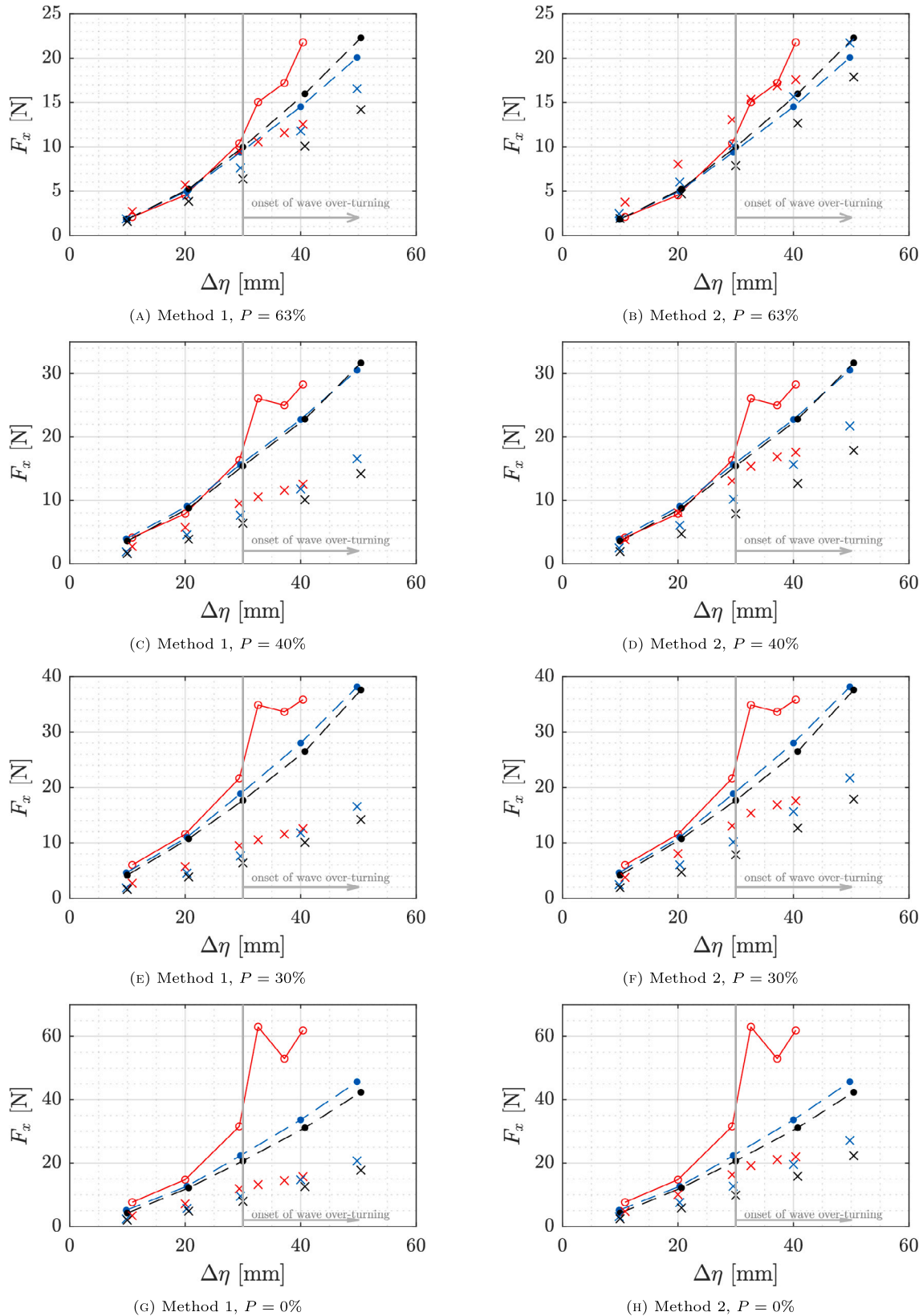


Fig. 5. The maximum WID loads associated with a subset of deterministic wave events focused on the front face of the topside structure; laboratory data compared to silhouette method predictions for topsides of different porosities, P . The silhouette method is based upon API recommendations (Method 1 on Table 2) and the updated procedure (Method 2 on Table 3). In all cases [—○—] corresponds to experimental data and [×] predicted values for $T_p = 1.4$ s. Likewise [—•—], [×] $T_p = 1.6$ s and [—×—], [×] $T_p = 1.8$ s.

This simplifies the application of the silhouette method. Each sub-plot concerns the maximum WID load, $F_{x,\max}^{\text{WID}}$, taken from a recorded time-history, $F_x^{\text{WID}}(t)$, addresses three spectral peak periods and shows its variation with the level of wave inundation, $\Delta\eta = \eta_c - h_d$. Within these cases the steepest waves ($\Delta\eta > 30$ mm for $T_p = 1.4$ s) exhibit some degree of wave breaking. This varies from incipient spilling to full over-turning depending on the local steepness Ak_p ; A being the linear amplitude sum, $A = \sum_{n=1}^N a_n(\omega)$ and k_p the wave number corresponding to the spectral peak. In particular it is evident that wave over-turning causes a significant increase in the measured loads in all cases. The point at which this occurs is indicated on the sub-plots of Fig. 5; the observations being consistent with earlier work and fully discussed in Ma and Swan (2020a). A total of eight sub-plots are provided, the four rows corresponding to topside structures with an effective porosity of $P = 63\%$, 40% , 30% and 0% (fully-plated). In each case the left-hand plot contrasts the laboratory data with the silhouette predictions (Eq. (1)) based upon Method 1 (Table 2) and the right-hand plot silhouette predictions based upon Method 2 (Table 3). In all cases an upper-bound solution for the drag coefficient C_d (Table 1) recommended by Finnigan and Petruskas (1997) and adopted in API (2014) has been applied.

Figs. 5(A) and 5(B), relating to $P = 63\%$, exhibit some important and easily explained trends. First, for small levels of inundation, $\Delta\eta$, the currently recommended application of the silhouette method (Method 1 on Fig. 5(A)) exhibits reasonable agreement with the laboratory data. However, for increased levels of inundation this model consistently under-predicts $F_{x,\max}^{\text{WID}}$. This is particularly apparent in cases involving breaking waves ($\Delta\eta > 30$ mm for $T_p = 1.4$ s) for which the largest measured load is as much as twice that predicted by the silhouette model (Method 1). The explanation for this is clear: the silhouette model is critically dependent upon an effective calibration of C_d . In seeking to achieve this Finnigan and Petruskas (1997) specifically noted that they excluded all breaking waves, arguing that they were not accurately reproduced in their wave basin. Having ignored what would undoubtedly have been the largest recorded loads, it is hardly surprising that the adopted calibration under-predicts the maximum measured loads. Similar arguments apply in respect of the updated silhouette model (Method 2 on Fig. 5(B)). In this case changes to the ‘equivalent’ regular wave, based upon additional non-physical iterations (including changes in the wave period as well as the water depth) produce an increase in the predicted water particle kinematics and hence an over-prediction of the maximum loads associated with small inundations. In contrast, the predicted loads associated with large inundations are closer to the measured data, when compared to Method 1, but they continue to under-estimate. Whilst the difference between the measured and predicted loads undoubtedly reduces with the application of Method 2, it is equally clear that it fails to predict the trend in the data (with $\Delta\eta$). Moreover, the largest loads on which reliability calculations will ultimately rely continue to be under-estimated.

Interestingly, with a reduction in the porosity of the topside structure, $P = 40\%$, 30% and 0% on Figs. 5(C,D) (E,F) and (G,H) respectively, the agreement between the measured and predicted loads gets progressively worse. Indeed, the measured data is under-estimated in all cases, for all levels of inundation, irrespective of the method of calculation employed. Likewise, the gross under-estimate of the maximum loads associated with breaking waves also increases; the measured loads being 2–3 times larger than those predicted using Method 2 in the worst (steepest) wave cases (Fig. 5(H)). Taken together, these changes confirm that the inadequacy of the silhouette method cannot be overcome by a simple re-calibration involving the definition of a larger drag coefficient; the value adopted in the present predictions being the maximum recommended in API (2014).

Drawing all the available data together (Table 4), Fig. 6(A) concerns Method 1 comparing the measured and predicted maximum loads as a function of $\Delta\eta$. Fig. 6(B) provides a related plot based upon Method 2.

In both cases it is clear that the measured loads are consistently under-estimated by the silhouette method. Further evidence of this is provided in Figs. 6(C) and 6(D). These again concern Method 1 and Method 2 respectively, with $F_{x,\max}^{\text{WID}}$ expressed as a function of the incident wave steepness, $\frac{1}{2}Hk$. Although, in these sub-plots, the trend in the data is perhaps less apparent than expected, the under-prediction is clear, occurring for all $\frac{1}{2}Hk$. This is further confirmed in Figs. 6(E) and 6(F) which directly compare the measured and predicted loads; the most significant errors arising in the largest loads generated by the steepest (breaking) waves and not necessarily involving the largest inundation, $\Delta\eta$. Taking Fig. 6 as a whole, the largest errors in the predicted loads is dependent upon both $\Delta\eta$ and $\frac{1}{2}Hk$. This indicates that the loading coefficients becoming increasingly unsuccessful as the incident waves diverge from the calibrated range, suggesting that neither the loading mechanisms assumed within the model (drag) nor the inputs to the model (steady or regular wave theory) are physically correct. The cumulative effect of both producing a significant under-prediction. In considering Figs. 5 and 6 the loads have been calculated using the API recommended procedures (Tables 2 and 3) with the loading coefficients set to the recommended maximum (Table 1).

4.2. Kaplan et al.’s (1995) model

Figs. 7 and 8 concerns the application of the Kaplan et al. (1995) WID load model. The first concerns an example load time-history, $F_x^{\text{WID}}(t)$, relating to one specific loading event for which $\Delta\eta = 30$ mm and $\frac{1}{2}Hk = 0.268$. In describing this model Kaplan et al. (1995) noted that the wave kinematics can either be predicted using a Stream Function solution (Dean, 1965) (appropriate to non-linear regular waves) or a linear random wave theory with Wheeler (1969) stretching. The data presented on Figs. 7(A) and 7(B) are closely related to these alternatives. Fig. 7(A) concerns a regular or steady wave modelled using the API recommended Method 2 (Table 3). In this case the difference between the measured and calculated wave elevations would be very similar to Fig. 2(A); the relevant lines being the Stokes 5th-order solution and the average wave profile. The measured and calculated maximum wave elevations are identical; the latter fitted to the former. However the overall wave profiles will differ significantly, as shown in Fig. 2(A). In contrast, Fig. 7(B) concerns an unsteady irregular wave modelled using the fully non-linear boundary element method (BEM) proposed by Hague and Swan (2009). This provides the best-possible description of both the wave shape, $\eta(x, y, t)$ and the water particle kinematics, $\mathbf{u}(x, y, z, t)$, high in the wave crest. Most importantly, it avoids the under-prediction commonly associated with Wheeler stretching (Fig. 3).

In both Figs. 7(A) and 7(B) three predicted load time-histories, $F_x^{\text{WID}}(t)$, are presented. The uppermost curve incorporates the porosity ($P = 63\%$) or solidity ($S = 1 - P = 0.37$) of the topside structure, but neglects any form of velocity blockage, both on the perimeter of the topside and within it. The inevitable result of this omission is that the measured data is over-predicted. In attempting to account for the velocity blockage Kaplan et al. (1995) argued that for individual structural members within the topside, the Keulegan–Carpenter number will be large (100–150) and consequently the fluid flow within the crest of the wave can be considered quasi-static, essentially a gradually varying current. Based upon this assumption, he employs the blockage factors proposed by Taylor (1991); the latter based on classical actuator disk theory and specifically developed for steady (time-independent) currents. This sub-divides the blockage into two parts: the initial reduction in the incident flow as it approaches the structure, due to the flow diversion around the structure, and the reduced velocities within the structure due to the cumulative sheltering of members/bodies located further upstream; the latter effect being closely associated with wake formation. Following Taylor (1991) the two blockage factors are defined by:

$$b_1 = \frac{1}{1 + \frac{C_d}{4}S} \quad \text{and} \quad b_2 = \frac{1 - \frac{C_d}{4}S}{1 + \frac{C_d}{4}S} \quad (6)$$

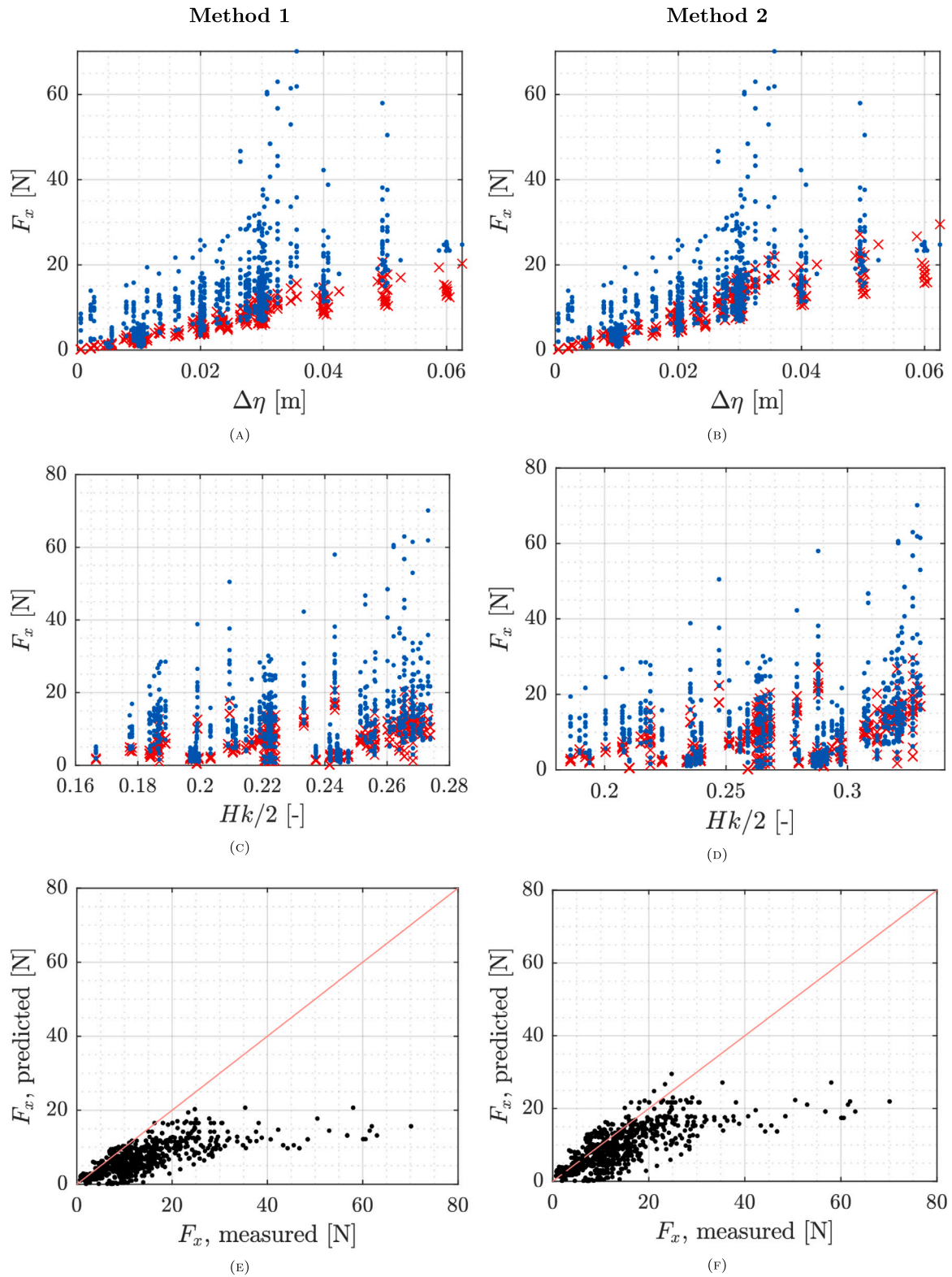


Fig. 6. The maximum WID loads, F_x^{wid} , from the full laboratory data base (Table 4) compared to the silhouette method: sub-plots (A,C,E) applied using Method 1 (Table 2), (B,D,F) using Method 2 (Table 3), with (A,B) expressed as a function of $\Delta\eta$, (C,D) a function of $Hk/2$ and (E,F) a direct plot of measured against predicted. [•] Experimental data, [×] silhouette method predictions and [•] a combination of both.

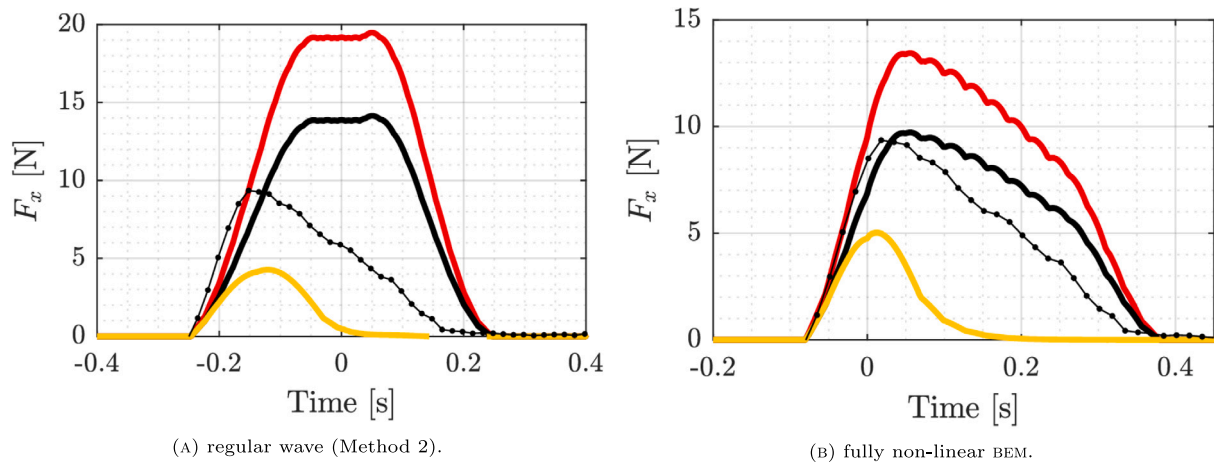


Fig. 7. An example WID loading time-history with comparisons to alternative implementations of the Kaplan et al. (1995) model. (A) Based on Method 2 (Table 3) for regular waves and (B) a fully non-linear BEM calculation of unsteady focused waves. On each sub-plot, [—] incorporates porosity (or solidity) but neglects any form of velocity blockage, [—] incorporates both solidity and velocity blockage such that $b_1 < 1$ and $b_2 = 1$, [—] incorporates solidity and velocity blockage where both $b_1 < 1$ and $b_2 < 1$; all compared to [—] experimental measurements.

For a topside structures represented by a series of porous grids, the velocity appropriate to the n -th grid would be $u_n = u_0 \cdot b_1 \cdot (b_2)^{n-1}$ where u_0 is the incident (unblocked) velocity, b_2 only acts from the second panel onwards and the effects on successive panels are cumulative (Kaplan et al. (1995) refers to a ‘cascade’ of blockage effects). Figs. 7(A) and 7(B) describe the changes in the predicted loads, $F_x^{\text{WID}}(t)$, due to the inclusion of first b_1 and then b_1 and b_2 ; the latter leading to an under-prediction of the loads irrespective of the kinematics model employed. Whilst Fig. 7(B) suggests that the inclusion of b_1 alone may give the improved agreement, subsequent results will show that this is simply fortuitous. Indeed, there is no justification for the inclusion of b_1 , but the neglect of b_2 . In practice, the difficulty lies with the gradually varying current approximation. This is at odds with the highly transitory and spatially localised nature of WID loading and appears inconsistent with Kaplan’s subsequent proposal to incorporate inertia and impact loading; the former dependent on the unsteady acceleration and the latter involving very rapid change. That said, the data presented on Fig. 7 does not preclude the possibility that the Kaplan model can provide accurate predictions rather than the input parameters, particularly the drag (C_d), blockage (b_1) and shielding (b_2) coefficients must be appropriately defined given both the nature of the loading and the kinematics employed. However, if this requires problem-specific empirical calibration, as appears to be the case, the model ceases to be truly predictive.

Fig. 8 concerns the most complete form of the Kaplan model, with solidity, drag, blockage and shielding all included using the recommended values. All of the loading events noted in Table 4 are included; the format of the figure being comparable to Fig. 6. The only difference is the applied kinematics models; the left column based upon API Method 2 (Table 3) and the right column the fully non-linear BEM, exactly as used in Fig. 7. Whilst the second approach undoubtedly provides an improved description of the incident waves, all of the comparisons (irrespective of the kinematics model employed) show that the Kaplan model consistently under-estimates the measured data. This is particularly apparent in Figs. 8(E) and 8(F); the largest errors again occurring in the steepest (breaking) wave cases involving a substantial but not necessarily the maximum deck inundation, $\Delta\eta$.

4.3. Graaf et al.’s (1995) model

Figs. 9 and 10 are the equivalent of Figs. 7 and 8, but with the models predictions based upon Graaf et al. (1995). In Fig. 9(A) the wave modelling was based upon the API recommended Method 2 (Table 3) appropriate to regular waves, whilst in Fig. 9(B) it was based upon

the fully non-linear BEM solution for unsteady or irregular waves. In both cases the time-history of the total WID load, $F_x^{\text{WID}}(t)$, is presented together with the two components contributing to this total; the face-load and base-load defined in Eqs. (3) and (4) respectively. In addition, the model predictions are compared to two sets of laboratory data. The first relates to a topside porosity of $P = 63\%$ is identical to that presented on Fig. 7. The second is a simplified solid box with the same overall geometry but zero porosity ($P = 0\%$). This latter case was chosen to be consistent with the assumption under-pinning the Graaf et al. (1995) model: that any momentum arising at the boundary of the topside structure is assumed to be instantaneously destroyed (Section 3.2).

The laboratory data presented on Fig. 9 confirms that with a reduction in P , the maximum WID load substantially increases. Nevertheless, if calculations are based upon a steady regular wave (Fig. 9(A)), the Graaf et al. (1995) model over-predicts both measured records. The explanation for this lies in two parts, both related to the use of an inappropriate (non-physical) wave model:

- The under-prediction of the fluid velocities high in the wave crest (Fig. 3) is more than offset by the increased volume of water (based on the wave shape, Fig. 2) arriving at the perimeter of the topside structure. This is expected to occur in all but the steepest breaking wave cases.
- With a (steady) regular wave neglecting all spatial evolution, the maximum crest-elevation and hence the maximum inundation extends over the entire plan area of the structure. This ensures that the face-load is maximised (subject to errors noted in (a)) and the based-load grossly over-predicted.

If, as an alternative, the Graaf et al. (1995) model is based upon a non-linear and unsteady wave model, the difficulties noted in (a) and (b) can be avoided. Evidence of this is provided on Fig. 9(B); the key point being the agreement between the model prediction and the $P = 0\%$ laboratory data. Indeed, it was this agreement, observed over a wide range of cases involving fully-plated ($P = 0\%$) topsides, that motivated the development of the Lagrangian Momentum Absorption (LMA) WID load model (Ma and Swan, 2020b); the challenge being to incorporate the porosity and layout of key elements/equipment within the topside structure.

Fig. 10 addresses all of the WID loading cases noted in Table 4; the maximum force, $F_{x,\text{max}}^{\text{WID}}$, again expressed in terms of both $\Delta\eta$ and $\frac{1}{2}Hk$, with direct comparisons between the measured and predicted loads demonstrating the extent of the over-prediction, as well as identifying a small number of cases in which the Graaf et al. (1995) model

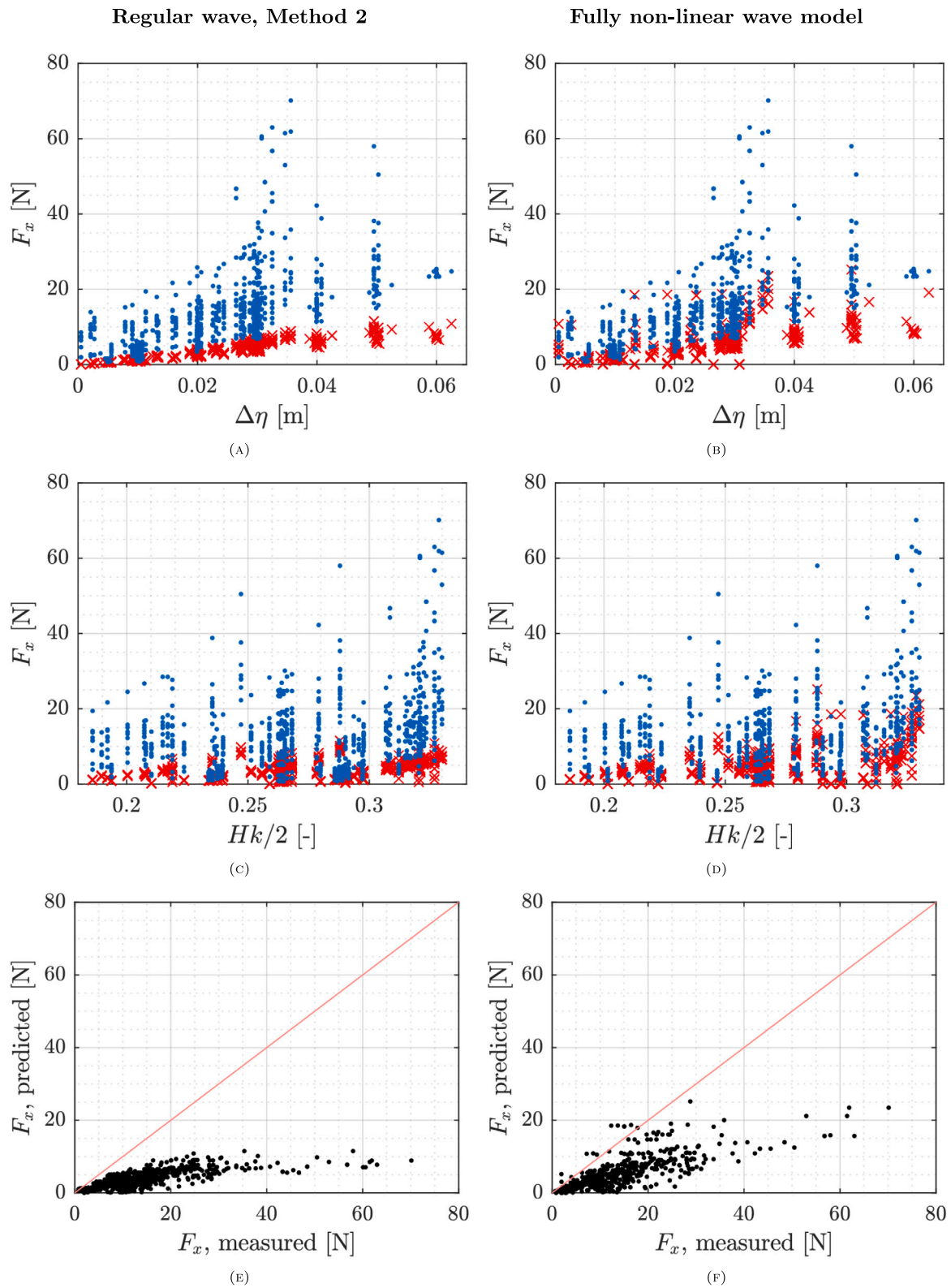


Fig. 8. The maximum WID loads, F_x^{wid} , from the full laboratory data base (Table 4) compared to Kaplan et al.'s (1995) model: sub-plots (A,C,E) applied using Method 2 (Table 3), (B,D,F) using fully non-linear BEM calculations of unsteady focused waves, with (A,B) expressed as a function of $\Delta\eta$, (C,D) a function of $Hk/2$ and (E,F) a direct plot of measured against predicted. [•] Experimental data, [×] predictions of the Kaplan model and [•] a combination of both.

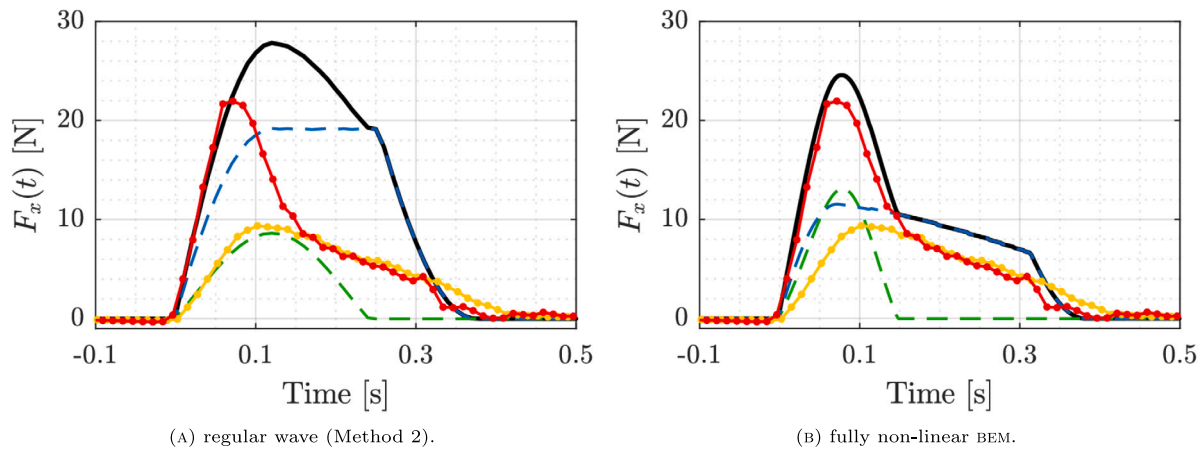


Fig. 9. Example WID loading time-histories with comparisons between the adoption of different wave models when calculating the loads based upon Graaf et al. (1995). (A) Using Method 2 (Table 3) for regular waves and (B) a fully non-linear BEM calculation of unsteady focused waves. Both sub-plots show measured time-histories on topsides of [—●—] 63% porosity and [—●—] 0% porosity, compared to [—] predicted load time-histories; the latter based upon Graaf et al. (1995) and separated into the [—] face-load and [—] base-load components.

becomes non-conservative. These involve very steep breaking waves (located where $\Delta\eta \approx 0.035\text{ m}$ and $Hk/2 \gtrsim 0.3$); the explanation being the increased rate of momentum transfer as a result of wave breaking (Ma and Swan, 2020a). Although significant improvements can be realised by applying a non-linear and unsteady wave model the solution remains, on average, over-predictive due to the neglect of topside porosity (Figs. 10(B,D)).

4.4. LMA (Ma and Swan, 2020b)

Having emphasised the importance of incorporating WID loading within a reliability analysis and having shown that the presently recommended WID load models do not yield accurate predictions without extensive and problem-specific empirical calibration (including the occurrence of breaking waves), it is important to emphasise that with the application of recently developed models all the required loading components can be successfully predicted. Fig. 11 provides evidence of this success; the LMA model (Ma and Swan, 2020b) being compared to the wide-ranging laboratory data considered earlier. Fig. 11(A) addresses the time-history of the individual loading event considered in Figs. 7 and 9 ($\Delta\eta = 30\text{ mm}$ and $\frac{1}{2}Hk = 0.268$). In this and all subsequent cases the wave theory on which the LMA load predictions are based is both fully non-linear and unsteady. This is an essential requirement of this load model. In the absence of any empirical calibration, or empirical coefficients of any form, the incident wave shape $\eta(x, y, t)$, its evolution in space (x, y) and time (t) (beneath the topside structure) and the associated water particle kinematics $u(x, y, z, t)$ must be accurately defined. As a result, the LMA model should not be applied using simplified regular waves (either Airy or Stokes waves) or linearised random/irregular wave theories; both will lead to large predictive errors. Having shown that the load time-history can be accurately predicted,¹ Figs. 11(B)–11(D) again concerns the full set of loading cases noted on Table 4. In contrast to Figs. 6, 8 and 10, it is clear that the LMA model provides a good description of the maximum measured loads, $F_{x,\max}^{\text{WID}}$, across a very broad range of loading events. This is most apparent in Fig. 11(D) which provides a direct comparison between the measured and predicted loads. Importantly, this figure confirms that the largest loads arising in the steepest (breaking) wave cases are equally well predicted. This demonstrates the advantage of applying

a relatively simple load model based upon accurate, fully-nonlinear, wave modelling; the latter being a pre-requisite for the success of the LMA model.

4.5. Topside orientation

Figs. 5–11 have considered the success of the available WID load models for varying levels of wave inundation ($\Delta\eta$), incident wave steepness ($\frac{1}{2}Hk$), and topside porosity (P). Whilst these are all key variables, the laboratory data base outlined on Table 4 also includes variations in the deck floor porosity (P_z) and the topside orientation ($\bar{\theta}$). Although P_z primarily affects the vertical loading, $\bar{\theta}$ will have a significant effect on the horizontal loads considered herein. Fig. 12 considers this; the success of the available models being considered in successive sub-plots. In each case the model predictions are compared to the measured data for increasing $\Delta\eta$ and four orientations of the topside front face, $\bar{\theta} = 90^\circ, 75^\circ, 60^\circ$ and 45° , where $\bar{\theta} = 0^\circ$ corresponds to the direction of wave propagation. Given the square plan area of the topside structure and the axi-symmetric layout of internal panels (Ma and Swan, 2020a) $90^\circ \geq \bar{\theta} \geq 45^\circ$ effectively covers the full range of all possible topside orientations. Whilst the data presented on Fig. 12 all relates to a topside porosity of $P = 40\%$, the comparisons are indicative of those occurring for all other porosities. Importantly, the under-prediction of the silhouette and Kaplan et al. (1995) models (Figs. 12(A), 12(B)), and the over-prediction of the Graaf et al. (1995) model for all values of $\bar{\theta}$ are consistent with both earlier results and the explanations provided. Interestingly, the variation with $\bar{\theta}$ depends upon whether the model is primarily driven by the wetted area (in the silhouette model) or the resolved component of the velocity in the Kaplan et al. (1995) and Graaf et al. (1995) models. This re-enforces the need for a model based upon both an accurate description of the incident waves and an appropriate loading mechanism. With the LMA model achieving both of these requirements the loads are successfully predicted irrespective of $\bar{\theta}$ (Fig. 12(D)).

4.6. Overall comparison

In an attempt to quantify the relative success of the available WID load models, Table 5 provides data relating to three commonly adopted model performance measures. These alternative values all seek to define the ‘goodness of fit’ to the measured data. To avoid a lengthy discussion of their relative merits, all three measures have been provided. With the modelling errors defined in terms of the difference in the maximum predicted and measured loads, both taken from a loading time-history, summed over all individual loading events ($i = 1$ to n), the three measures are defined by:

¹ Whilst this agreement is very good, it only represents one example. Many other examples are provided in Ma and Swan (2020b) and Ma and Swan (2023); the former addressing a broad range of incident wave conditions and the latter widely varying topside configurations.

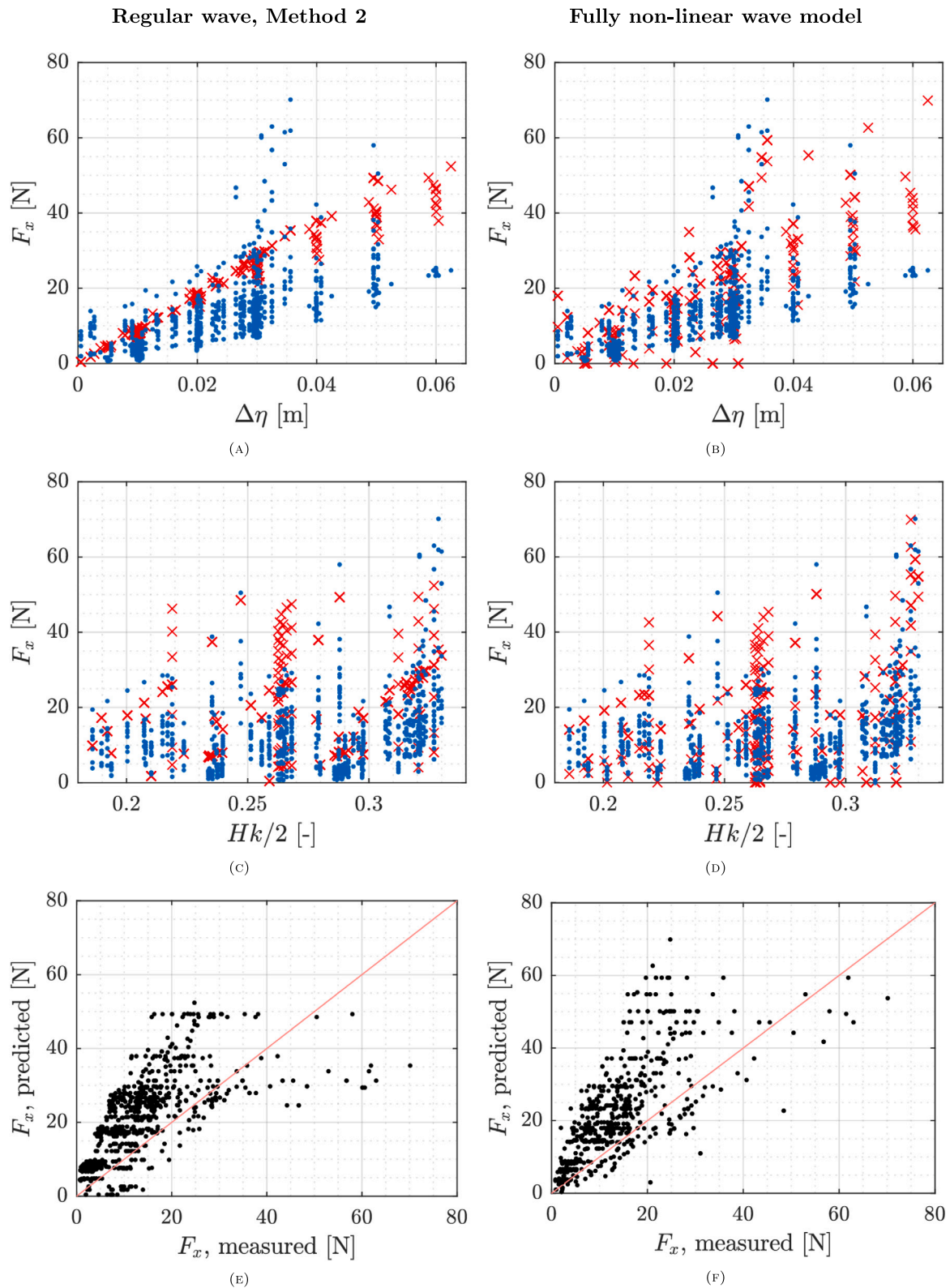


Fig. 10. The maximum WID loads, F_x^{wid} , from the full laboratory data base (Table 4) compared to Graaf et al.'s (1995) model: sub-plots (A,C,E) applied using Method 2 (Table 3), (B,D,F) using fully non-linear BEM calculations of unsteady focused waves, with (A,B) expressed as a function of $\Delta\eta$, (C,D) a function of $Hk/2$ and (E,F) a direct plot of measured against predicted. [•] Experimental data, [×] predictions of the Graaf et al. (1995) model and [•] a combination of both.

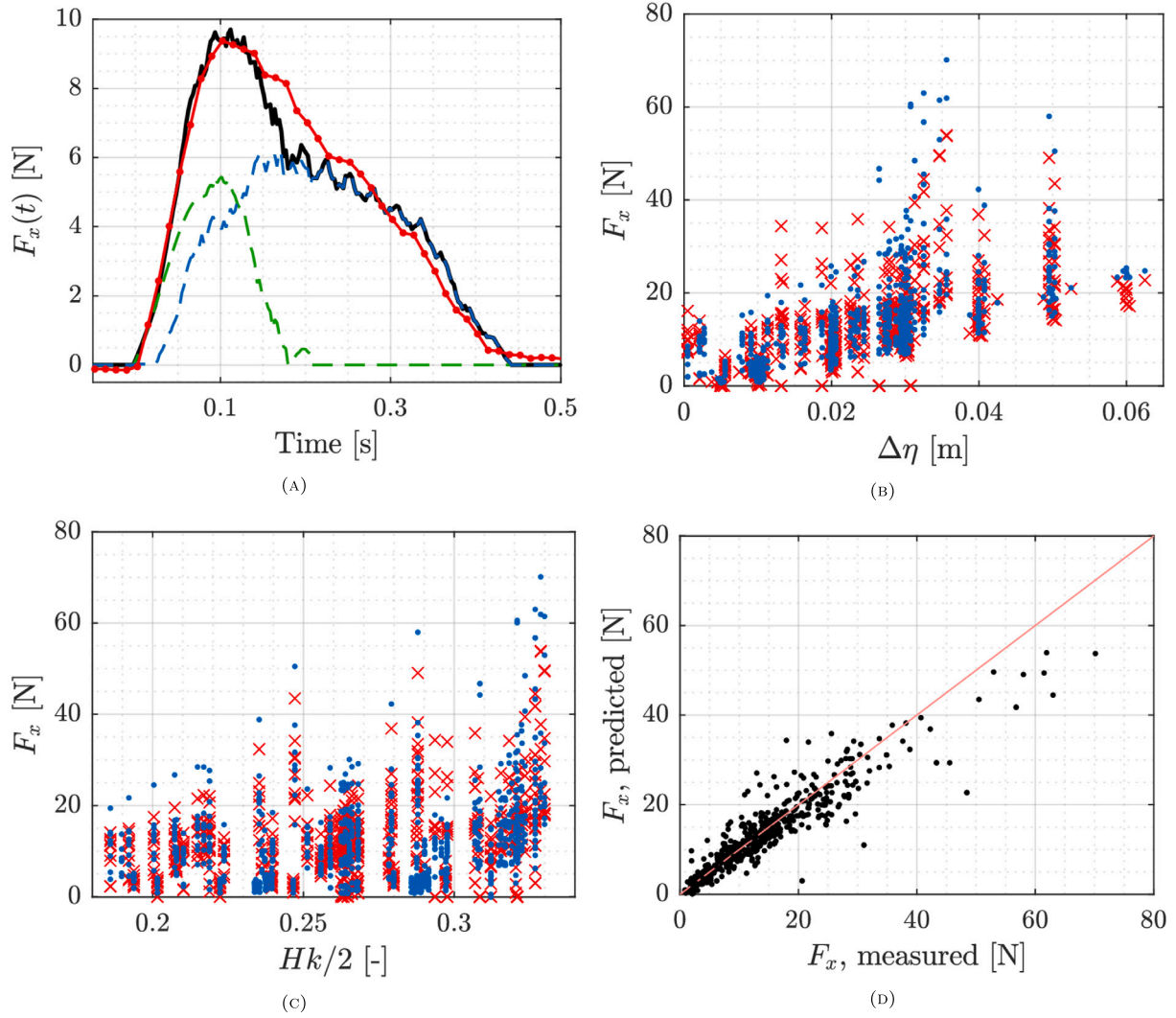


Fig. 11. Comparisons to the LMA model (Ma and Swan, 2020b) based upon fully non-linear BEM calculations of unsteady focused waves. (A) An example loading time-history $F_x^{\text{wid}}(t)$: [—•—] experimental measurement compared to [—] the predicted load time-history; the latter separated into the [—] face-load and [—] base-load components. (B,C,D) The maximum WID loads, $F_{x,\text{max}}^{\text{wid}}$ from the full laboratory data base (Table 4) with (B) expressed in terms of $\Delta\eta$ and (C) in terms of $Hk/2$ and (D) a direct plot of measured against predicted: [•] experimental data, [×] LMA (Ma and Swan, 2020b) predictions and [•] a combination of both.

(i) Mean square error (MSE) =

$$\frac{1}{n} \sum_{i=1}^n (F_{x,\text{max}}^P - F_{x,\text{max}}^M)^2 = \frac{1}{n} \sum_{i=1}^n (\Delta F_i^2) \quad (7)$$

(ii) Root mean square error (RMSE) =

$$\sqrt{\text{MSE}} \quad (8)$$

(iii) Median absolute-deviation (MAD) =

$$\text{median} \langle |\Delta F_1|, |\Delta F_2|, |\Delta F_3|, \dots, |\Delta F_n| \rangle \quad (9)$$

where the super-script ‘P’ denotes predicted and ‘M’ measured. In addition to these values, the fourth row on Table 5 defines the mean deviation (MD). Whilst this is not an effective measure of model performance (or fit) it confirms whether (on average) the model over-predicts or under-predicts; the former perhaps being acceptable in a re-assessment context, the latter not.

In considering the values of (i), (ii) and (iii) given on Table 5 it is important to note that they simply seek to quantify the predictive ability of the model over a very broad range of loading events. Since not all of these events will be relevant to all topside structures (for example, increases in the deck elevation, h_d , will limit the extent of any inundation, $\Delta\eta$, for a given exceedance probability), the values

given on Table 5 do not directly relate to success/accuracy in design calculations. Nevertheless, they are indicative of this; the key points given as follows:

- The silhouette method typically under-predicts the maximum measured WID load. This occurs irrespective of the adopted wave modelling (Tables 2 and 3); the updated API recommendations (Santala, 2017) providing a small improvement based upon all 3 measures of modelling success. This simply arises because of an increase in the predicted kinematics and should not be taken to imply the appropriateness of either the load model (drag) or the adopted wave theory (regular Stokes waves).
- The (Kaplan et al., 1995) model also under-predicts. This can be, to some extent, improved by the adoption of a fully non-linear unsteady wave model, but the model success remains poor. Interestingly, if the effects of shielding (b_2^{n-1}) on the internal panels is neglected, the model over-predicts by a comparable amount. This confirms the fortuitous nature of the apparent agreement between the Kaplan model (with $b_2 = 1.0$) and the measured data on Fig. 7(B). Moreover, it demonstrates the sensitivity of the Kaplan model to the chosen coefficients, a significant difficulty without problem-specific data to effectively calibrate the model.

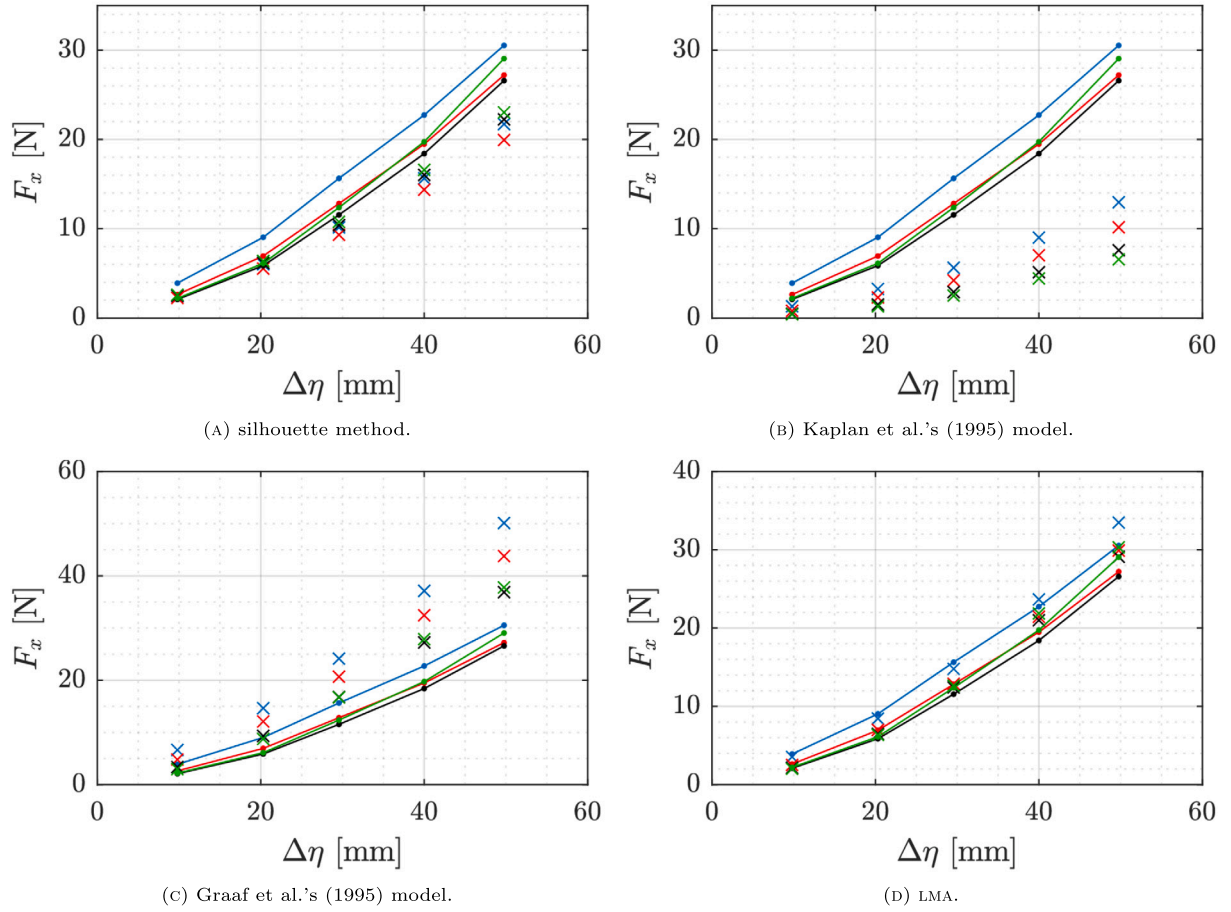


Fig. 12. Comparisons between the measured and predicted maximum loads for varying topside orientations based upon different models for [×, —●—] $\bar{\theta} = 90^\circ$, [×, —●—] 75° , [×, —●—] 60° and [×, —●—] 45° where [—●—] corresponds to experimental data and [×] model predictions. All cases relate to $P = 40\%$.

Table 5

Model performance measures based on Eqs. (7), (8) and (9); providing alternative quantification of the 'goodness of fit'. Additional data describing the mean deviation (MD) is also provided to identify under-estimates and over-estimates.

	Silhouette method		Kaplan et al. (1995)			Graaf et al. (1995)		LMA
	Method 1	Method 2	Method 2	BEM ($b_2 = 1.0$)	BEM	Method 2	BEM	BEM
MSE [N ²]	97.27	66.14	166.75	119.6	147.85	132.87	164.85	17.29
RMSE [N]	9.86	8.13	12.91	34.58	12.16	11.53	12.84	4.16
MAD [N]	20.37	10.10	64.70	7.71	46.46	65.22	61.78	2.09
MD, ΔF [N]	-6.29	-4.13	-9.51	3.35	-8.15	7.41	8.18	-0.82

- The Graaf et al. (1995) model over-predicts the measured data. In this case the adoption of an improved kinematics model produces no noticeable improvement, irrespective of the measure adopted, (i), (ii) or (iii). This suggests that (realistic) increases in the fluid velocities (Fig. 3) are in large part offset by changes in the predicted wave shape and that the single most important issue is the failure to account for the porosity or openness of the topside structure.
- Finally, the LMA model provides, by far, the best fit to the maximum measured loads, $F_{x,\max}^{\text{wid}}$. This is supported by all three measures of modelling success; the quantified deviations or errors being almost an order of magnitude smaller. Most importantly, it should be emphasised that this modelling success is achieved using a very simplistic but complete load model (essentially momentum transfer based upon Newton's second law), a realistic, fully-nonlinear description of the incident waves and no empirical coefficients/calibration.

5. Concluding remarks

This paper has provided a quantitative assessment of the commonly adopted models used to describe the wave-in-deck loads that arise when the height of an incident wave-crest exceeds the deck elevation and the topside structure experiences direct wave loading. The difficulty of incorporating such events within a full structural reliability analysis, particularly where it relates to the required re-assessment of an older structure with a lower deck elevation, is explained. The implications of recent advances in the description of extreme waves, not least the non-linear increases in crest-elevation (beyond second-order), the increased fluid velocities high in a wave crest, and the occurrence of wave breaking, add to these difficulties. The required changes to the design/re-assessment process are both direct, in terms of a need to accurately describe the increased loads, and indirect in terms of the need to consider a broader range of sea-states and individual wave

loading events. This latter point raises concerns regarding any pre-determined 'design point'. In many instances the key to these challenges lies in the accurate prediction of the WID loads.

To investigate this a large laboratory data base of WID loading events provide the benchmark against which present recommended practice from the international design standards and guidance notes, recently proposed updates to these methods/models, and the present state-of-the-art in terms of both wave modelling and WID load predictions are compared. The results arising from this extensive set of comparisons confirm that none of the WID load models currently recommended in the international standards/guidance notes provide an adequate representation of the applied loads. Moreover, in several instances it is recommended that these models are applied in conjunction with a non-linear regular or steady (Stokes) wave. In such cases the wave characteristics at high elevations within the wave crest (the only part relevant to WID loading) are so far removed from the physical reality of extreme waves in realistic seas that extensive and problem-specific calibration becomes an essential requirement. This is not what would be expected of a predictive load model, adding significantly to the cost and duration of a full reliability analysis, particularly where this requires the investigation of multiple sea-states to define potential 'worst-case' loading events and their respective return probabilities.

Having shown that present recommended practice has significant limitations in respect of WID load predictions, comparisons to the same laboratory data base has confirmed the broad success of the recently proposed Lagrangian Momentum Absorption (LMA) WID load model (Ma and Swan, 2020b). This adopts a simplistic, but complete, formulation of the applied loads (essentially a transfer of momentum from the wave crest into the topside structure following Newton's second law), is capable of incorporating the topside layout and fully non-linear description of the incident waves. Provided that the incident wave conditions are accurately modelled both in terms of the wave shape and the underlying water particle kinematics, the model is accurate, requires minimal computational effort, and is not dependent on any empirical coefficients or calibration. As such, it is entirely appropriate to design/re-assessment calculations.

CRedit authorship contribution statement

Li Ma: Conceptualization, Methodology, Software, Validation, Formal analysis, Investigation, Data curation, Writing – original draft, Writing – review & editing, Visualization, Project administration. **Chris Swan:** Conceptualization, Methodology, Resources, Writing – original draft, Writing – review & editing, Supervision, Project administration.

Declaration of competing interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

Data availability

Data will be made available on request.

References

- API, 2014. Structural Integrity Management of Fixed Offshore Structures API RP 2SIM. American Petroleum Institute (API), Washington D.C.
- Bateman, W.J.D., Swan, C., Taylor, P.H., 2001. On the efficient numerical simulation of directionally-spread surface water waves. *J. Comput. Phys.* 174, 227–305.
- Bateman, W.J.D., Swan, C., Taylor, P.H., 2003. On the calculation of the water particle kinematics arising in a directionally spread wave field. *J. Comput. Phys.* 186, 70–92.
- Boccotti, P., 1983. Some new results on statistical properties of wind waves. *Appl. Ocean Res.* 5 (3), 134–140.
- Chen, Y., Wu, Y., Bahuguni, A., Gullman-Strand, J., Lv, X., Lou, J., Ren, W., 2018. Directional wave-in-deck loading on offshore structures with porous and plated decks with supporting I-beams. *Coast. Eng.* 137, 79–91.
- Dean, R.G., 1965. Stream function representation of nonlinear ocean waves. *J. Geophys. Res.* 70 (18), 4561–4572.
- DNV-GL, 2019. Recommended Practice DNVGL-RP-C205: Environmental Conditions and Environmental Loads. DNV-GL, Norway.
- Dommermuth, D.G., Yue, D.K.P., 1987. A high-order spectral method for the study of nonlinear gravity waves. *J. Fluid Mech.* 184, 267–288. <http://dx.doi.org/10.1017/S002211208700288X>.
- Efthymiou, M., van de Graaf, J.W., Tromans, P.S., Hines, I.M., 1997. Reliability-based criteria for fixed steel offshore platforms. *J. Offshore Mech. Arct. Eng.* (ISSN: 0892-7219) 119 (2), 120–124. <http://dx.doi.org/10.1115/1.2829053>.
- Ewans, K.C., 1998. Observations of the directional spectrum of fetch-limited waves. *J. Phys. Oceanogr.* 28 (3), 495–512. [http://dx.doi.org/10.1175/1520-0485\(1998\)028<0495:OOTDSO>2.0.CO;2](http://dx.doi.org/10.1175/1520-0485(1998)028<0495:OOTDSO>2.0.CO;2).
- Fenton, J., 1985. A 5th-order Stokes theory for steady waves. *J. Waterw. Port Coast. Ocean Eng.* 111 (2), 216–234.
- Finnigan, T., 1991. Prediction of Wave Forces on Decks of Offshore Platforms. Technical Report TM91000025, Chevron Oil Field Research Report.
- Finnigan, T.D., Petruskas, C., 1997. Wave-in-deck forces. In: ISOPE, Vol. 3. Golden, CO, USA, pp. 19–24.
- Forristall, G.Z., 2000. Wave crest distributions: Observations and second-order theory. *J. Phys. Oceanogr.* 30 (8), 1931–1943. [http://dx.doi.org/10.1175/1520-0485\(2000\)030<1931:WCDOAS>2.0.CO;2](http://dx.doi.org/10.1175/1520-0485(2000)030<1931:WCDOAS>2.0.CO;2).
- Gibson, R., Swan, C., 2007. The evolution of large ocean waves: the role of local and rapid spectral changes. *Proc. R. Soc. Lond. Ser. A Math. Phys. Eng. Sci.* 463 (2077), 21–48.
- Graaf, J.W.v.d., Tromans, P., Vanderschuren, L., 1995. Wave loads on decks. *Shell Offshore Struct. Eng. Newsl.* (10).
- Hague, C.H., Swan, C., 2009. A multiple flux boundary element method applied to the description of surface water waves. *J. Comput. Phys.* 228 (14), 5111–5128.
- Hasselmann, K., Barnett, T., Bouws, E., Carlson, H., Cartwright, D., Enke, K., Ewing, J., Gienapp, H., Hasselmann, D., Kruseman, P., et al., 1973. Measurements of Wind-Wave Growth and Swell Decay During The Joint North Sea Wave Project (JONSWAP). Technical Report, Deutsches Hydrographisches Institut.
- Heideman, J.C., Weaver, T.O., 1992. Static wave force procedure for platform design. In: *Civil Engineering in the Oceans V*. ASCE, pp. 496–517.
- ISO, 2013. Petroleum and Natural Gas Industries — Fixed Steel Offshore Structures (ISO 19902:2007/Amd.1:2013). International Organization for Standardization (ISO).
- Kaplan, P., Murray, J.J., Yu, W.C., 1995. Theoretical analysis of wave impact forces on platform deck structures. In: OMAE 1995, 14th International Conference on Offshore Mechanics & Arctic Engineering, Vol. 1, No. A. ASME, Copenhagen, Denmark, p. 189.
- Karpadakis, I., Swan, C., 2020. On the average shape of the largest waves in finite water depths. *J. Phys. Oceanogr.* (2020).
- Karpadakis, I., Swan, C., 2022. A new crest height distribution for nonlinear and breaking waves in varying water depths. *Ocean Eng.* (ISSN: 0029-8018) 266, 112972. <http://dx.doi.org/10.1016/j.oceaneng.2022.112972>, URL: <https://www.sciencedirect.com/science/article/pii/S0029801822022557>.
- Karpadakis, I., Swan, C., Christou, M., 2019. Laboratory investigation of crest height statistics in intermediate water depths. *Proc. R. Soc. Lond. Ser. A Math. Phys. Eng. Sci.* 475 (2229), 20190183. <http://dx.doi.org/10.1098/rspa.2019.0183>, URL: <https://royalsocietypublishing.org/doi/abs/10.1098/rspa.2019.0183>.
- Latheef, M., Swan, C., 2013. A laboratory study of wave crest statistics and the role of directional spreading. *Proc. R. Soc. Lond. Ser. A Math. Phys. Eng. Sci.* 469 (2152).
- Latheef, M., Swan, C., Spinneken, J., 2017. A laboratory study of nonlinear changes in the directionality of extreme seas. *Proc. R. Soc. Lond. Ser. A Math. Phys. Eng. Sci.* 473 (2199), 20160290.
- Lighthill, J., 1979. Waves and hydrodynamic loading. In: *Proceedings of the Second International Conference on the Behaviour of Off-Shore Structures*, Held At Imperial College, London, England.
- Lindgren, G., 1970. Some properties of a normal process near a local maximum. *Ann. Math. Stat.* 1870–1883.
- Ma, L., Swan, C., 2020a. An experimental study of wave-in-deck loading and its dependence on the properties of the incident waves. *J. Fluids Struct.* (ISSN: 0889-9746) 92, 102784. <http://dx.doi.org/10.1016/j.jfluidstruct.2019.102784>.
- Ma, L., Swan, C., 2020b. The effective prediction of wave-in-deck loads. *J. Fluids Struct.*
- Ma, L., Swan, C., 2023. An experimental study of wave-in-deck loading and its dependence on the properties of the topside structure. *Mar. Struct.* (ISSN: 0951-8339) 88, 103340. <http://dx.doi.org/10.1016/j.marstruc.2022.103340>, URL: <https://www.sciencedirect.com/science/article/pii/S0951833922001769>.
- Morison, J., O'Brien, M., Johnson, J., Schaaf, S., 1950. The forces exerted by surface waves on piles. *J. Pet. Technol. Petroleum Transactions, AIME* 189, 149–154.
- NORSOK, 2017. N-003 Actions and Action Effects, third ed. Norwegian Technology Standards Institution (NORSOK), Mustads vei 1, NO-0283 Oslo, Norway.
- Santala, M.J., 2017. Resolving the API RP 2MET crest conundrum for wave-in-deck loading. In: *Offshore Technology Conference*. Chevron Energy Technology Company.
- Sharma, J.N., Dean, R.G., 1981. Second-order directional seas and associated wave forces. *Soc. Petrol. Eng. J.* 21 (1), 129–140.
- Swan, C., Latheef, M., 2016. Extreme surface water waves in realistic directionally spread seas. In: *Extreme Events and Rogue Waves Seminar*. Bad Honnef, Germany.

- Taylor, P., 1991. Current blockage: Reduced forces on offshore space-frame structures. In: All Days. OTC, <http://dx.doi.org/10.4043/6519-ms>.
- Tromans, P.S., Anaturk, A.R., Hagemeyer, P., 1991. A new model for the kinematics of large ocean waves-application as a design wave. In: The First International Offshore and Polar Engineering Conference. International Society of Offshore and Polar Engineers.
- Tychsen, J., Risvig, S., Hansen, H.F., Hansen, N.-E.O., Stevanato, F., 2016. Summary of the impact on structural reliability of the findings of the tyra field extreme wave study 2013-15. In: 3rd Offshore Structural Reliability Conference. pp. 14–16.
- van Raaij, K., Gudmestad, O.T., 2007. Wave-in-deck loading on fixed steel jacket decks. *Mar. Struct.* 20, 165–184.
- West, B.J., Brueckner, K.A., Janda, R.S., Milder, D.M., Milton, R.L., 1987. A new numerical method for surface hydrodynamics. *J. Geophys. Res.: Oceans* 92 (C11), 11803–11824. <http://dx.doi.org/10.1029/JC092iC11p11803>, URL: <https://agupubs.onlinelibrary.wiley.com/doi/abs/10.1029/JC092iC11p11803>.
- Wheeler, J.D., 1969. Methods for calculating forces produced by irregular waves. *Offshore Technol. Conf. (OTC-1006-MS)*, <http://dx.doi.org/10.4043/1006-MS>,