Review of Wave in Deck Load Assessment Procedure
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HEALTH & SAFETY EXECUTIVE
MaTSU/8781/3420

REVIEW OF
WAVE-IN-DECK LOAD
ASSESSMENT PROCEDURES

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BOMEL Report No. C681104V040R Revision C
ACKNOWLEDGEMENTS

This study compares alternative methods for calculating wave loads in deck structures. A number of the models have been developed in-house by Operators who have generously made the information available to this study. Particular acknowledgement is made to the following people and organisations who have contributed information and given their time for discussions:

- J M Gebara, Amoco Worldwide Engineering & Construction
- T Finigan and C Petrauskas, Chevron Petroleum Technology Company
- R G Harwood, Shell UK Exploration and Production
  and J W van de Graaf, SIPM
- J I Dalane, Statoil

It is through the contributions of these people and organisations that this comprehensive comparison has been possible.

Particular acknowledgement is made of the contribution of the late Paul Kaplan of Hydromechanics Inc. He provided specific input to this project but more widely his work has given industry the basis to understand and model the complex effects of wave loading on the decks of offshore structures.
SUMMARY

This document reports on work concerning wave-in-deck load assessment methods for fixed offshore structures. The work has been performed for the Offshore Safety Division of the UK Health and Safety Executive (OSD-HSE) by Billington Osborne-Moss Engineering Limited (BOMEL) as the main contractor in conjunction with subcontractor Offshore Design a.s. and Dr Paul Kaplan of Hydromechanics Inc.

The objectives of the study are:

- To provide a review of the state-of-the-art in predicting wave-in-deck loads,
- To assess their potential significance for North Sea structures in terms of system reliability, and
- To compare the applicability of different calculation methods that have been used to date.

Uncertainties in crest height or structural response calculations are not considered in detail and emphasis is placed on the methods by which incident deck forces are calculated.

The study is introduced in the context of historic regulatory requirements and the increasing focus on system reliability reflecting different environmental circumstances. Particular concerns regarding wave-in-deck loads beyond the existing design envelope have been precipitated by subsidence under North Sea fields and hurricanes in the Gulf of Mexico which have damaged older platforms with low deck heights.

The experimental programmes which have resulted are reviewed, both to illustrate the physics of wave-in-deck loads and to give some insight to the conditions for which theoretical models have been developed and calibrated. Six different models are then considered with approaches which span from detailed component modelling through to a simple wetted area projection. The API RP2A Section 17 model is in the public domain and the Kaplan formulation is available with proprietary software. The Statoil approach has been published in outline and Chevron first published their method in 1997. The remaining models due to Amoco and Shell have been made available for the purposes of this study.

This document provides a qualitative comparison of the methods, complete with quantitative examples in terms of calculated loads and reliability implications. Backed by other results in the literature, recommendations for calculating wave-in-deck loads are made.

Overall it is found that the model selection and uncertainty are less significant than recognition of the need to account for wave-in-deck loads at all. It is important that the limitations due to the empirical basis of the models are recognised so that an inappropriate bias is not introduced. For simple jacket reliability calculations for the North Sea, there are certain practical advantages in adopting the Shell model. The model is based on the gross horizontal flux into a control volume enclosing the deck and topsides, is readily programmable, and is independent from case specific calibrations. For more detailed or critical evaluations the greater sophistication of the Kaplan model may be appropriate. This appears to be the most accurate model from a theoretical standpoint, is perhaps the most extensively calibrated, includes all important effects and the software, although proprietary, is commercially available. For Gulf of Mexico conditions Chevron's models provide well validated methodologies.
Most importantly the study has highlighted that for many North Sea structures, failure due to environmental overload could only be associated with wave loading in the deck. It is therefore essential that system reliability evaluations take due account of these loads and their influence on collapse modes. This report provides new information to enable meaningful calculations to be undertaken.
CONTENTS

1. INTRODUCTION 1
   1.1 BACKGROUND 1
   1.2 SCOPE OF WORK 3
   1.3 REPORT LAYOUT 3

2. DEFINITIONS 5

3. REGULATORY REQUIREMENTS AND ASSOCIATED GUIDANCE ON AIR GAP 7
   3.1 HSE GUIDANCE 8
   3.2 API RECOMMENDED PRACTICE 9
   3.3 NORWEGIAN PRACTICE 10
   3.4 DRAFT ISO STANDARD 11
   3.5 CONCLUSION 11

4. PHYSICS OF WAVE-IN-DECK LOADING 13
   4.1 WAVES 13
   4.2 HYDRODYNAMIC LOADING 14
   4.3 DECK CONSTRUCTION 15
   4.4 OFFSHORE OBSERVATIONS 16

5. EXPERIMENTAL STUDIES OF DECK FORCES 19
   5.1 EKOFISK TEST PROGRAMME 19
   5.2 GULF OF MEXICO TEST PROGRAMME 20
   5.3 OTHER TEST PROGRAMMES 22
<table>
<thead>
<tr>
<th>Section</th>
<th>Title</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>6.</td>
<td>REVIEW OF MODELS</td>
<td></td>
</tr>
<tr>
<td>6.1</td>
<td>API RP2A SECTION 17</td>
<td>25</td>
</tr>
<tr>
<td>6.1.1</td>
<td>Model Description</td>
<td>25</td>
</tr>
<tr>
<td>6.1.2</td>
<td>Model Enhancement</td>
<td>27</td>
</tr>
<tr>
<td>6.2</td>
<td>KAPLAN MODEL</td>
<td>27</td>
</tr>
<tr>
<td>6.2.1</td>
<td>Model Description</td>
<td>27</td>
</tr>
<tr>
<td>6.2.2</td>
<td>Correlation between Theory and Experiment</td>
<td>28</td>
</tr>
<tr>
<td>6.2.3</td>
<td>Modified Procedure and Results for Double-bottom Structure</td>
<td>30</td>
</tr>
<tr>
<td>6.3</td>
<td>SHELL'S WAVE-IN-DECK MODEL</td>
<td>31</td>
</tr>
<tr>
<td>6.3.1</td>
<td>Introduction</td>
<td>31</td>
</tr>
<tr>
<td>6.3.2</td>
<td>Basic Theory</td>
<td>32</td>
</tr>
<tr>
<td>6.4</td>
<td>STATOIL APPROACH</td>
<td>36</td>
</tr>
<tr>
<td>6.5</td>
<td>CHEVRON APPROACH</td>
<td>37</td>
</tr>
<tr>
<td>6.6</td>
<td>AMOCO APPROACH</td>
<td>39</td>
</tr>
<tr>
<td>6.7</td>
<td>SUMMARY</td>
<td>42</td>
</tr>
<tr>
<td>7.</td>
<td>QUANTIFIED COMPARISON OF THE VARIOUS WAVE-IN-DECK LOAD MODELS</td>
<td>45</td>
</tr>
<tr>
<td>7.1</td>
<td>MODEL SELECTION</td>
<td>45</td>
</tr>
<tr>
<td>7.2</td>
<td>AMOCO SECTION 17 COMPARISON</td>
<td>46</td>
</tr>
<tr>
<td>7.3</td>
<td>WAVE-IN-DECK LOAD CALCULATIONS</td>
<td>48</td>
</tr>
<tr>
<td>7.3.1</td>
<td>API RP2A Section 17 - example calculations</td>
<td>50</td>
</tr>
<tr>
<td>7.3.2</td>
<td>Shell method - example calculations</td>
<td>54</td>
</tr>
<tr>
<td>7.3.3</td>
<td>Kaplan model - example calculations</td>
<td>57</td>
</tr>
<tr>
<td>7.3.4</td>
<td>Statoil method</td>
<td>63</td>
</tr>
<tr>
<td>7.4</td>
<td>COMPARISON OF EXAMPLE CALCULATIONS</td>
<td>65</td>
</tr>
</tbody>
</table>
8. IMPLICATIONS OF WAVE-IN-DECK LOADS FOR STRUCTURAL RELIABILITY
   8.1 PREVIOUS STUDIES 73
   8.2 CASE STUDY RELIABILITY CALCULATIONS 77
   8.2.1 Reliability Model 77
   8.2.2 Model Response Predictions 79
   8.2.3 Reliability Analysis Results 81
   8.2.4 Conclusions from the Case Study 86

9. CONCLUSIONS 87

10. REFERENCES 91

APPENDIX 95
1. INTRODUCTION

1.1 BACKGROUND

Consideration of wave impact forces on offshore structures has been a subject of concern for platform designers during the past 20 years. The earliest interest was for impact forces on horizontal members (circular cross-section) of a platform in the "splash zone", as illustrated by both analytical and experimental studies which included measurements on a large-scale test structure in the Gulf of Mexico\(^{(1,3)}\). While such effects were relatively localised on smaller elements of a platform, more significant forces were found to occur where impact on significant components such as platform decks was encountered.

When the first generation of fixed offshore platforms were designed through to the 1960s knowledge was limited regarding wave heights, wave forces and the variability of environmental conditions with time. Consequently the air gaps of many of the older platforms are less than their present day counterparts. Instead of large waves with associated long return periods passing underneath the decks of these platforms, some waves may impact the deck resulting in large local and global loads for which the deck-jacket interface and supporting structure were not designed.

In assessing the safety of existing platforms in extreme events, it is essential that the potential significance of wave-in-deck loads be accounted for. It is important to recognise that even a small exceedance of the air gap can generate significant loads with considerable uncertainty thereby having a major influence on risk and reliability\(^{(6,25)}\). Wave loading on decks often results from the occurrence of more extreme waves than were originally estimated for design purposes, but can also arise from underestimating or overlooking foundation subsidence and reservoir compaction, changes in codified procedure reflecting more onerous design criteria, interaction of large waves on structural components, and wave steepness outside the design assumptions. Instances in the Ekofisk region of the Norwegian sector have precipitated major experimental and theoretical developments on the topic.

Individual operators such as Phillips in Norway have undertaken major test programmes\(^{(13)}\) to address specific concerns in the Ekofisk region. The findings have been applied in rigorous structural reliability analyses which, by comparison with API RP2A methods\(^{(37)}\), have underlined the need for careful selection of calculation procedures appropriate to the North Sea. Beyond the initial evaluation of hydrodynamic loads, alternative methods, for example to account for blockage effects and energy dissipation as the crest passes through the structure, are being proposed by other authors\(^{(26)}\). The purpose of the present study is to bring these methods together and consider their applicability for North Sea practice.

Although instances of subsidence of the seabed have raised specific concerns in the North Sea, the effects of wave loading in the deck are sufficiently severe and credible\(^{(6,27)}\) for it to be imperative that the hazard is explicitly considered, particularly in relation to older structures in general where reserve strength ratios (a measure of strength beyond that provided for in the design condition) may be low and as-built air gaps may be inadequate to obviate large waves reaching the deck. Suitable goal setting guidance has been given in the UK\(^{(40)}\).

To demonstrate sufficient reliability, the long return period extreme waves being considered may encroach the deck such that the resulting loads and associated uncertainties must be rationally accounted for. However, wave-in-deck loads have not been considered routinely in North Sea ultimate strength assessments where the approach has been to factor the 50-
or 100-year environmental loading profile in a linear manner. It has been custom and practice to neglect any potential for the waves to reach the deck, relying on the air gap to provide a margin of safety. However, in light of present day knowledge, the historic value of this air gap (1.5m) for all locations and design criteria cannot be considered to provide a rational or consistent basis for assuring safety. Hence, it is imperative for reserve strength calculations to take account of the actual configuration and realistic loading scenarios. By considering the combination of waves and currents associated with a particular return period and modelling wave loads in the deck where appropriate meaningful measures of system reliability can be derived.

A cautionary example may be given where a Reserve Strength Ratio (RSR) of 3 was calculated but, once wave-in-deck loads were evaluated, the estimated annual probability of failure was unacceptable necessitating extensive platform strengthening\textsuperscript{(22)}. The assessment of two platforms in the Gulf of Mexico, using the new API RP2A Section 17 methodology\textsuperscript{(9)}, found that wave-in-deck loads can contribute some 30% to the overall base shear, even in cases where grating rather than plating formed the deck\textsuperscript{(23,24)}.

API RP2A Section 17 was developed to address concerns regarding the ageing population of US platforms and specifically includes a procedure for wave-in-deck force calculations reflecting experience of hurricane damage or catastrophic collapse in the Gulf of Mexico\textsuperscript{(25,26)}. Section 17 has further exposed the limitations of the existing design process and has increased the awareness of wave-in-deck loads. However, before the approach is adopted into UK practice, it is appropriate to determine when and how the effects need be accounted for, particularly to ensure all regulatory requirements are met.

The API formulation for calculating wave-in-deck loads is empirical, based on limited tests of representative Gulf of Mexico structures. The US Gulf of Mexico environment is rather different from that in the UKCS. In the Gulf of Mexico wave heights are generally lower and wave-in-deck problems result from infrequent hurricane passages rather than more regular severe storms. Furthermore, Gulf of Mexico structure designs are often dominated by static loads, whereas in the North Sea fatigue loads have been a major consideration. Finally the transparency of North Sea structures to waves may differ and the significance of neglecting vertical loads in the API formulation needs investigation. It was recognised that the adoption of API Section 17 for North Sea applications in light of these differences needed to be investigated and on this basis the present study was instigated.

Birkinshaw\textsuperscript{(23)} had previously highlighted the air gap issue as being an important factor in the assessment of structural reliability. He recognised considerable uncertainties associated with estimating this type of load:

- determination of the height and profile of the impacting crest;
- accurate prediction of the load transmitted to the structure through accurate hindcast / environmental data;
- prediction of the response of the structure.

It is reasonable to expect that the level of these uncertainties needs to be reduced if reliable estimates of the loads associated with air gap exceedance and their contribution to reducing structural reliability are to be determined. The dominant uncertainties are to be identified in the present study so that future investigations are directed appropriately.
1.2 SCOPE OF WORK

On the basis of the foregoing background, the present study was established to draw together industry approaches for calculating wave forces on deck structures. The overall aim was to identify methods by which the wave loads associated with air gap erosion can be accounted for in ‘jacket’ reliability analyses. In achieving this goal, it was intended to improve general understanding of how loads are calculated and characterised in assessment methods and how deck loads can influence the overall structural reliability. Specific tasks were as follows:

- Collation of available models / data from both the public domain and previously confidential sources;
- Review and appraisal of models;
- Quantified comparison of selected methods;
- Consideration of implications for structural reliability;
- Reporting.

1.3 REPORT LAYOUT

This report for the study begins in Section 2 with a definition of terms for airgap evaluation. Regulatory requirements for air gap provision in past, present and future guidelines are set down in Section 3 by way of background to current concerns. Section 4 presents a baseline explanation of the mechanisms of load generation from wave-structure interaction. The test programmes on which most of the theoretical models depend for calibration are summarised in Section 5, before Section 6 provides an in-depth review of the numerical modeling approaches. Detailed explanations are backed by summary comparisons from which the advantages and disadvantages of their adoption for use in North Sea jacket assessments are determined. A specific case study and parametric variations are introduced in Section 7 where deterministic comparisons are generated forming input to reliability evaluations in Section 8. Conclusions and recommendations for future practice are delivered in Section 9.

The Appendix to the report includes an evaluation of air gap trends indicated by the HSE fixed steel platform database(3). The results give some insight to the variable significance of wave-in-deck loading for North Sea structures based on historic practice and to the need for a more rational approach to controlling the risks in future.
2. DEFINITIONS

In undertaking this review it has appeared that the basis and language for calculating air gap differs between authors and the purpose of this section is to set down some baseline definitions against which the present work may be interpreted.

Figure 2.1 overleaf provides an annotated illustration of air gap and the associated environmental influences. The terms defined by Figure 2.1 as used variously in different guidance documents are:

- Crest height
- Airgap
- Wave height
- Storm still water level
- HAT (MHHW)
- Storm surge
- Mean sea level
- LAT (MLLW)
STORM STILL WATER LEVEL
WAVE PEAK (S)
WAVE HEIGHT (T)
CREST HEIGHT (F)
STORM SURGE (E)
MEAN SEA LEVEL
RAW TEXT

NOTES:
1. Extensive Wave Height = Maximum Individual Wave Height with N Year Return Period (Calculation Methods Differ).
2. Storm Surge = Tidal Change Due to Wind and Atmospheric Pressure Differential.
4. HAT = Highest Astronomical Tide; (MMH) = Mean Highest High Water
5. LAT = Lowest Astronomical Tide; (MLW) = Mean Lowest Low Water
6. Wave Period - In Practice Has Been to Use a Range of Periods to Reflect Steepness of 1:14 to 1:20.
   Shorter Waves (More Steepness) Will Increase the Crest Height and Current Tends
   Associated with the Use of One Wave Height May Warrant Further Consideration
7. Crest Height = Use of Crest Height Models and Statistics is Still in the Research Stage
   and Has Not Currently Been Adopted into Design Practice

Figure 2.1
Air gap calculation and environmental influences
3. REGULATORY REQUIREMENTS AND ASSOCIATED GUIDANCE ON AIR GAP

Present day regulations and guidelines for offshore installations recognise the significance of the air gap and include alternative design provisions to ensure wave action either does not impinge on the 'topsides' or else is adequately accounted for. Different regional approaches from the UK (HSE\(^{46,48}\)) and Norway (NPD\(^{49}\)) are reviewed below and compared with the draft ISO standard\(^{30}\) which is seeking to evolve a harmonised state-of-the-art position.

Table 3.1 provides a comparison of the provisions but for a fuller explanation reference should be made to the descriptions which follow. This review is important for understanding the context in which the wave-in-deck load calculation methods need to be applied.

<table>
<thead>
<tr>
<th>Code</th>
<th>Basic Provision</th>
<th>Additional Considerations</th>
</tr>
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</table>
| HSE\(^{48}\) (Present Regulation) | Goal setting  
- no specific air gap requirement  
- effects may be accounted for as a performance standard based on structural and operational considerations. |                                                                                           |
| HSE\(^{46}\) (Previous Guidance) | Prescriptive requirement for air gap above 50yr extreme crest \(\geq 1.5\)m to account for:  
- possibility of extreme waves | Additional consideration to be given to:  
- diffraction (for \(D/L > 1/2\))  
- settlement  
- uncertainty in determining water depth and extreme crest elevation |
| API\(^{15}\) | Air gap above 100yr design wave crest \(\geq 5\)ft \((1.5)\)m to account for:  
- settlement  
- uncertainty in determining water depth  
- possibility of extreme waves | Additional consideration to be given to:  
- known or predicted long term subsidence |
| ISO\(^{30}\) (Draft) | Air gap above 100yr design wave crest to be based on appropriate reliability and experience accounting for:  
- settlement  
- uncertainty in determining water depth  
- possibility of extreme waves and  
Air gap \(\geq 1.5\)m or larger regional requirement | Additional consideration to be given to:  
- known or predicted long term sea-floor subsidence |
| NPD\(^{49}\) | Air gap to be sufficient for the probability of structural damage due to extreme waves to be less than \(10^{-4}\) per year.  
- no absolute value  
- all uncertainties to be accounted for | None - all eventualities covered under basic provision |
3.1 HSE GUIDANCE

In reviewing the HSE Guidance Notes\textsuperscript{50}, it is important to recognise that compliance has never been a legal requirement in the UK. The regulatory position is given in the safety case regulations (SCR)\textsuperscript{44} and the Offshore Installations and Wells (Design and Construction etc) Regulations (DCR)\textsuperscript{45}, replacing the Statutory Instrument 289\textsuperscript{43} which the Guidance Notes supported. Although the onus is now on the Duty Holder to use good engineering practice to demonstrate that an installation “can withstand such forces acting on it as are reasonably foreseeable”, the Guidance Notes provide a baseline in terms of industry experience based on the air gap strategy. It is important to understand the basis and limitations of these familiar provisions in the context of risk and reliability\textsuperscript{46}.

It should be noted that the Guidance Notes\textsuperscript{50} are based on the provision of minimum 50 year criteria. The new goal setting regulations and interpretive guidance have replaced this proposed minimum by reasonable forces. No return period is prescribed. This has the effect that Guidance must be seen as a specific example of a 50 year calculation procedure.

Section 10.2 of the Guidance Notes\textsuperscript{50} entitled “Clearance above waves (air gap)” states the following: “That part of the superstructure which is not designed to resist wave impact should have a clearance ‘air gap’ above the design extreme wave crest. In assessing the air gap, reference should be made to the extreme environmental conditions set out in Section 11 [of the Guidance Notes].”

Section 11.1.2 - Environmental considerations, states that “An accurate assessment of the meteorological and oceanographic environment is fundamental to the sound design of offshore installations. Only on this basis is it possible to calculate the loads likely to be imposed by natural phenomena and to predict the behaviour of structures under extreme loading conditions and prolonged exposure”.

Section 10.2 further states “During the lifetime of the structure, there is a significant probability that the design extreme wave crest, based on a 50-year return period will be exceeded. The air gap should therefore be based on an assessment of the probability of encountering extreme crests of return period greater than 50 years, and in addition the air gap relative to the design extreme crest elevation should never be less than 1.5m”. The implication is that the 1.5m allowance is sufficient to account for the probability that the extreme wave crest will be exceeded in the lifetime of the structure.

The guidance recommends that allowance should be made for the effects of settlement, and uncertainties in estimating water depth and extreme crest elevation. Specific reference is made to diffraction effects associated with large diameter members (D/L >\textsuperscript{1/3}) which may increase the wave crest level.

A procedure for calculating the extreme surface elevation is given in Section 11.10.2 of the Guidance along with default values for the metocean parameters which can be used if site-specific values are not available. The extreme surface $E_{50} = d_m + L_{50} + C_{50}$

where  

- $d_m =$ the mean water depth
- $L_{50} =$ 50-year extreme still water level variation due to tidal and surge variations
- $C_{50} =$ 50-year extreme wave crest elevation.

Although reference is made to the extreme surface elevation with a return period of 50 years, as joint probabilities are only accounted for in the determination of the 50-year extreme still water level ($L_{50}$), $E_{50}$ is in excess of 50 years, and may be greatly in excess. Appendix A15 of the Guidance Notes also provides limited information on methods to
determine possible loads on items in the air gap; as follows:

- In lieu of more detailed analysis, it may be assumed that the particle kinematics in the air gap are the same as for the crest of a 50-year return period wave.
- The local effects of fluid loading on all items in the air gap should be considered using the factor of safety appropriate for storm conditions.
- Global effects of fluid load on items in the air gap need only be considered if the items are substantial in relation to the main structural members immediately below the air gap. In such cases reduced safety factors may be used provided the probability of structural failure is kept to an acceptable level.

It will be seen through this report that this Guidance should be adopted with caution in light of the better understanding now available concerning the significance of wave crest properties and the potential implications for system reliability for even small excursions of wave loading in the deck. The future of this Guidance and of the document in general is under active consideration by HSE as UK regulator.

### 3.2 API RECOMMENDED PRACTICE

The US methodology for determining air gap, ‘deck clearance’ is presented in Section 2.3.4g of API RP2A-WSD\(^5\) and Section C.3.6 of API RP2A-LRFD. In both it is stated that: *“Large forces result when waves strike a platform’s deck and equipment. To avoid this, the bottom of the lowest deck should be located at an elevation which will clear the calculated crest of the design wave with adequate allowance for safety. Omni-directional guideline wave heights with a nominal return period of 100 years, together with the applicable wave theories and wave steepnesses should be used to compute wave crest elevations above storm water level, including guideline storm water level, and guideline storm tide. A safety margin, or air gap of at least 5ft [-1.5m] should be added to the crest elevation to allow for platform settlement, water depth uncertainty and for the possibility of extreme waves in order to determine the minimum acceptable elevation of the bottom beam of the lowest deck to avoid waves striking the deck. An additional air gap should be provided for any known or predicted long term seafloor subsidence.”* As stated by Smith and Birkinshaw\(^6\), the exact definition of the guideline storm tide is unclear, however, it appears to have a value which is expected to be close to the 100-year extreme still water elevation.

The 1.5m air gap allowance in API RP2A caters for unexpected platform settlement, water depth uncertainty and crest heights in excess of those associated with the design condition. The air gap is prescribed irrespective of site conditions and therefore gives variable safety levels when compared with UK HSE Guidance\(^8\). The UK Guidance air gap allowance accounts for crest height variance alone stating that due consideration should be given additionally to unexpected platform settlement and water depth uncertainty\(^8\) (see Table 3.1). Thus it would appear that the API RP2A guidance is less conservative compared with UK HSE Guidance.

For the densely populated Gulf of Mexico zone of US waters, RP2A provides a translation of the air gap provisions as a graph of minimum deck height (above mean lowest low water) (see Figure 3.1).
This simple translation reflects the very specific and well documented conditions in the region. In other areas, individual calculations are necessary.

Within Section 17 of RP2A, where higher probabilities of failure for existing platforms than new design have been deemed acceptable by the code drafters to a degree dependent on consequence of failure, similar curves are presented reflecting the shorter return period of the assessment event. In this context the curves are used to determine whether inundation of the deck may occur and, if so, to act as a trigger requiring ultimate strength level checks to be carried out.

In addition, the commentary to Section 17 provides a specific method for calculating wave loading on the deck in the event of inundation. This is examined in detail in Section 6.1 of this report but the inclusion of guidelines should be noted as a significant change from other standards. The API guidelines are the most recent of those reviewed here and the inclusion of wave-in-deck requirements reflects the recent understanding of the potential implications for platform reliability.

### 3.3 NORWEGIAN PRACTICE

In the Norwegian Petroleum Directorate’s offshore regulations (1994)\(^9\), a different approach to air gap determination is prescribed than that required by either traditional UK or US practice. The approach is based on a limit state methodology, and requires that extreme waves do not cause major structural damage with an annual probability which exceeds 10\(^-4\). It is considered acceptable for local damage to occur, provided that the structure remains capable of withstanding the 100-year environmental load without progressive collapse. In
both the 10,000-year (intact) and 100-year (damaged) scenarios, load and resistance factors of unity are to be used.

No additional air gap allowance (similar to the 1.5m used in UK and US practices) is required. The implication is that issues such as settlement and water depth uncertainty will be accounted for explicitly in the determination of the 10,000-year condition to which the structure is to be subjected.

It should be noted that Norwegian practice appears to be to define ‘air gap’ as the elevation of the underdeck above mean water level rather than above the storm crest as indicated in Figure 2.1. This distinction needs to be recognised when reviewing the literature.

3.4 DRAFT ISO STANDARD

The harmonisation procedure used in developing the draft ISO fixed steel structures standard\(^{(30)}\) has used the provisions of API RP2A LRFD as a starting point. With respect to air gap, it is not yet clear whether the requirements will remain similar to RP2A but provision for other approaches is likely to be included with suitable text such as ‘the safety margin [air gap] should be based on appropriate reliability considerations and experience’ against which 1.5m may be given as an overriding minimum (Section C.6.3.6.1.2 of Draft A). However, it is clear that regional requirements will have to be accommodated.

The Commentary has been drafted giving alternative reliability approaches and methods for crest height calculation to enable worldwide application. The absence of a definitive recommended approach at this stage reflects the fact that research is ongoing on both technical and methodological issues. The continuing debate similarly confirms the increasing recognition that the consequences of wave inundation can be severe and proper account must therefore be taken.

At present the ISO draft standard retains the API RP2A Section 17 wave-in-deck calculation methodology for which crest heights determined from wave theories are considered adequate.

3.5 CONCLUSION

This brief review of the treatment of air gap in regulations, codes and standards has shown that UK and US practices have historically been similar, dictating a finite margin between the wave crest and deck. By contrast, Norwegian standards have set a more rational requirement that air gap should be sufficient for target platform reliabilities to be achieved. The draft international standard is moving to adopt this approach, reflecting the inconsistent safety margin delivered historically by adopting the same air gap in different environments. The regional variation of wave heights and storm tides with return period and the consequent erosion of air gap was illustrated by Smith and Birkinshaw for the North Sea\(^{(30)}\). Table 3.2 reproduces their results demonstrating the relatively short return period associated with erosion of the 1.5m air gap in the Northern North Sea compared with the Southern sector. The variation gives an order of magnitude less reliability for Northern North Sea structures with air gap designs to the same absolute provisions.
Table 3.2
Return Periods associated with $E_{50}$ and air gap exceedence ($E_{50}$)

<table>
<thead>
<tr>
<th>Location</th>
<th>$E_{50}$ (m)</th>
<th>$E_{50}$ Return Period</th>
<th>Air Gap Return Period ($E_{50} + 1.5m$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>NNS</td>
<td>19.6m</td>
<td>800 years</td>
<td>3,800 years</td>
</tr>
<tr>
<td>CNS</td>
<td>14.9m</td>
<td>600 years</td>
<td>&gt;10,000 years</td>
</tr>
<tr>
<td>SNS</td>
<td>8.8m</td>
<td>850 years</td>
<td>&gt;&gt;10,000 years</td>
</tr>
</tbody>
</table>

A supplementary investigation to examine the historic air gap provision for UK sector platforms was annexed to the present study and is reported in Appendix A. The apparent trend, as the form of the HSE guidance may imply, was for air gaps to be spread across a broad but consistent range. As indicated by Smith and Birkinshaw, the concern is that this delivers inconsistent, and unknown, reliability.

Further evidence was gathered by BOMEL for a group of platforms selected at random from similar locations. The available design data were used in conjunction with HSE proposed extrapolations for long term environmental statistics to give a first pass estimate of zero air gap return periods. The results are presented for illustrative purposes in Figure 3.2. The general conclusions from Appendix A are that there has been no significant change in air gap provision in platform design over the last three decades and, as confirmed by Figure 3.2, the reliability which has been delivered varies significantly even for platforms in close proximity.

The overriding conclusion must be that whilst the 1.5m minimum air gap provision has attempted to deliver a margin of safety, it does not provide a rigorous or consistent safety margin. As with all low probability high consequence events, there is no data to support the adequacy of this margin.

![Airgap variation with return period for different SNS platforms](image)

Figure 3.2
Air gap variations in UK sector platforms for the same region
4. PHYSICS OF WAVE-IN-DECK LOADING

In Section 6 a range of models for calculating loads due to waves in the deck of offshore structures is reviewed. The models give different representations of wave loading and are inconsistent (judgement based) in the effects they include and neglect. In essence all methods, whilst attempting to provide a practical model of the complex fluid-structure interactions that take place, are necessarily providing a pragmatic compromise between theoretical exactness and empirical evidence. The balance for each method is different; some are based on holistic approaches whilst others are based on summation of discrete forces.

In order to compare and contrast the models meaningfully, it is appropriate to consider first a simplistic representation of the mechanics involved. For more detailed information, reference can be made to Barltrop and Adams\textsuperscript{(3)} or the HSE Guidance Notes\textsuperscript{(7)}.

4.1 WAVES

Ocean waves are generated by wind. In reality, ocean waves are irregular, being made up of superimposed waves; nevertheless for practical purposes they are modelled by regular or periodic wave theories. As the overall wave profile propagates, the water particles describe 'circular' orbits, with smaller orbits and hence lower velocities away from the surface.

The surface profile and particle velocities at different spatial positions at different stages through the wave cycle can be calculated from available wave theories. The linear Airy wave theory is simple to apply but neglects terms included for example in Stoke's higher order theories. Including these terms captures the asymmetry between the crest and trough in a given wave.

Figure 4.1 taken from Barltrop and Adams\textsuperscript{(3)} and adopted in API RP2A\textsuperscript{(7)} was developed for the determination of environmental loads. Although not specifically structured to determine the best representation of wave kinematics, the figure can be used to indicate the regions for which different wave theories may be considered appropriate. It is recognised that some theories are good at predicting wave profile and others at predicting internal kinematics.

The degree of asymmetry determined using the different theories varies significantly. Furthermore, when calculating deck loads due to inundation by the wave crest in the context of system reliability calculations, it is the statistics of crest height occurrence which are applicable. Adopting nonlinear wave theories alone to account for the asymmetry in a given wave implies (incorrectly) a unique correlation between wave height, crest height and hence force. More sophisticated methods based on hindcast data\textsuperscript{(6,30)} have been developed to provide a better representation of the physical and statistical characteristics. However this is an area of considerable uncertainty.
4.2 HYDRODYNAMIC LOADING

To calculate the hydrodynamic forces acting on elements of an offshore structure the temporal and spatial effects need to be considered which include\(^8\):

- Drag forces
- Inertial forces
- Lift forces (vortex shedding cycle)
- Slam forces
- Slap forces
- Pressure
- Spatial effects dependent on \(K_c\).

The nature of the forces involved can be summarised as follows:

- Drag forces are caused by viscosity resulting in flow separation.
- Inertial forces are related to the acceleration of the incident flow and the modification of the incident wave pattern by the member. When the element is small with respect to the wavelength there is little effect on the wave pattern and Morison's equation (with the convective term neglected) has been demonstrated to be an appropriate representation:

\[
F = F_d + F_i = \frac{1}{2} C_d \rho U |U| + C_m \rho A \ddot{U}
\]  
(4.1)
where:

\[ F \] is the instantaneous combined drag plus inertia force per unit length resolved normal to the member axis (N/m)

\[ F_d \] is the instantaneous drag force per unit length resolved normal to the member axis (N/m)

\[ F_i \] is the instantaneous inertia force per unit length resolved normal to the member axis (N/m)

\[ C_d \] is the drag coefficient

\[ \rho \] is the density of water (kg/m³)

\[ D \] is the member diameter or characteristic cross-sectional dimension (m)

\[ |U| \] is the modulus of the instantaneous velocity of the incident flow resolved normal to the axis of the member (m/sec)

\[ U \] is the instantaneous velocity of the incident flow resolved normal to the axis of the member (m/sec)

\[ C_m \] is the inertia coefficient

\[ A \] is the member cross-sectional area (m²)

\[ \dot{U} \] is the instantaneous acceleration of the incident flow resolved normal to the axis of the member (m/sec²)

- Slam forces occur when a wave engulfs a member causing a volume of water to be decelerated.
- Slap forces are similar to slam forces but refer to a breaking wave front meeting an approximately parallel member.

### 4.3 DECK CONSTRUCTION

Offshore topsides structures worldwide differ significantly depending on the form of construction. Examples are as follows:

- Closed modules presenting opaque walls to the outside face / Open deck structures with exposed equipment and piping
- Plated deck with underdeck beams and stringers with vertical and horizontal faces exposed / Doubled bottomed deck with opaque lower face
- Deck plates / Deck grating.

Clearly the ability for waves to penetrate the deck and the magnitude and direction of loads generated varies significantly depending on the construction. Figure 4.2 provides a simplistic representation in which the vertical and horizontal ‘transparency’ can be considered separately.
Figure 4.2
Wave impact on decks and topsides equipment

In the first case it is clear that opaque plating provides a barrier to the flow such that significant forces might be expected as the wave steepens, reflects and diffracts. In the second case the vertical loads must be considerably less in the absence of a base plate and with only deck grating; however downwards as well as upwards forces can be generated as the surface elevation falls. With an open deck structure, waves may be incident on internal equipment but with shielding and disruption to the flow. If these loads are not accounted for in the performance standards, they may trigger other hazards with consequent implications for safety. Furthermore lateral loads may be generated on the base beams at different points through the structure as the wave passes.

The permutations are too many to explore here; nevertheless this simple overview serves to underline the difficult task in identifying an appropriate wave-in-deck load calculation model for all circumstances. Furthermore it emphasises the importance of ensuring that empirical data used for calibration are relevant to the circumstances to which the models are being applied.

4.4 OFFSHORE OBSERVATIONS

Some understanding of the nature of wave-in-deck effects has been gathered from offshore observations. Kvitrud[41] presented a catalogue of ten deck inundations for fixed jacket, semisubmersible and TLP installations in the Norwegian sector between 1981 and 1995. The effects included variously damage to the deck, burner boom and secondary structures.
as well as movement of equipment and impairment of services. Although several of the affected platforms were in the Ekofisk region where subsidence has eroded the air gap, waves also encroached the temporary deck of the modern (1994) Europipe 16/11 jacket. The associated instrumentation confirmed that the crest height corresponded to the 10,000-year expectation and was associated with considerable asymmetry; Smith and Birkinshaw\textsuperscript{49} report that the crest within the 26m wave was 19m.

More specific information about the interaction of the waves with the structure was obtained by Bothelo et al\textsuperscript{49} from a survey of damage to 30 platforms in the South Timbalier region of the Gulf of Mexico, following the passage of Hurricane Andrew. The specific objective was to document any consistent and defensible physical evidence of damage caused by wave forces to deck equipment or topside structural members.

The survey uncovered three consistent pieces of physical evidence of topside wave force damage on many of the structures:

- Bent handrails, stairways and, in some cases, (bent) heavy deck support beams on the cellar deck level.
- Severely bent swing rope support beams (4" S-shaped beams) installed over boat landings on the east side of many of the structures.
- Steel framed well head signs installed on opposite sides of each well head and bent in line with the wave direction.

Less consistent evidence was indicated for the movement or upending of heavy (>2000lb) equipment and deck hatches.

It is interesting to note that much of the damage catalogued was localised. There was, in the opinion of Bothelo et al, no evidence of a ‘wall of water’ moving through the platform at a given elevation. Damage due to water impact at the higher elevation was very localised. The authors\textsuperscript{49} found that in many instances the south end of the cellar or subcellar deck was severely damaged with no damage at the north end. In other instances the water reached the main decks without causing substantial damage.

The physical evidence (damage) was used to infer the maximum water elevation or, alternatively, the maximum wave height the structure may have been exposed to during the hurricane. It was impossible to determine by how much the water level exceeded a certain elevation and hence a lower bound estimate was calculated based on ½ of the wave between the two ‘wetted’ deck elevations. The upper bound for the range was calculated based on the maximum ‘wetted’ deck elevation. A comparison of the inferred and hindcast wave crest height distributions for both upper and lower bounds of crest height is shown in Figure 4.3, indicating that the hindcast data are in good agreement with the physical evidence. This was then used in hindcast calculations to validate platform collapse / survival predictions, using for example the API RP2A wave-in-deck loading methodology\textsuperscript{45}.

It should be noted that the hindcast crest height is based on the Stream Function wave theory with a 7% increase as a device to reflect better the wave asymmetry (see also Finnigan and Petrasauskas\textsuperscript{43}). Furthermore the results of the platform collapse analyses were not always consistent with the post Hurricane Andrew observations; of the thirteen platforms examined, two survived the hurricane although the hindcast analysis ‘predicted’ failure and conversely one platform failed although the analysis indicated it should have survived\textsuperscript{49}. Ultimate strength analysis is complex and the disparity may be influenced by the structural analysis techniques, uncertainties associated with the foundation or k-bracing failure modes involved, as well as uncertainties in the crest height experienced and method for calculating wave-in-deck loads.
Figure 4.3
Comparison of Hurricane Andrew hindcast wave crest predictions

The potential significance of deck inundation can be appreciated from another example where wave-in-deck loads contributed 30% of the overall base shear.

The experiences provide useful insight to the mechanisms of crest penetration and damage for these generally open structures in the hurricane environment.
5. EXPERIMENTAL STUDIES OF DECK FORCES

Experimental studies of wave-in-deck forces have generally been precipitated by concerns in specific regions. In particular the population of platforms in the Gulf of Mexico includes many older installations designed for 25-year conditions with inadequate clearance for protection against hurricane waves. The consequences have been seen in recent hurricanes and investigations into the loading and resistance have been undertaken to form a rational basis for safe operating decisions. In other areas, such as the Ekofisk region of the North Sea, significant subsidence of the seabed has brought the platform decks closer to the sea increasing the probability of inundation. Additional structures such as the Ekofisk barrier have been built to provide a degree of protection, in that case to the deck of the Ekofisk tank. However reflected waves from the barrier acted to increase wave heights, exacerbating conditions for the jackets in the diffracted field. To provide further protection, the Ekofisk jack-in project in 1987 raised the jacket decks by 6m. Nevertheless continuing subsidence has remained a concern. It is to be recognised that aspects of the associated test programmes have been directed towards specific situations.

This section gives an overview of test work, directed at these two regions, as the results have formed the basis for calibrating the available models for wave-in-deck load calculation. More general programmes are noted in Section 5.3.

5.1 EKOFISK TEST PROGRAMME

Early studies of wave forces on deck structures motivated by the concerns about the effects of wave forces on Ekofisk platforms were carried out at Norwegian Hydrodynamic Laboratories in 1985. Further experimental studies and measured force results for a particular Ekofisk platform (2/4C) were presented in Reference 11, together with some associated hydrodynamic and structural analysis studies.

These early tests demonstrated that effects due to impact forces were definitely present in the measured force time histories, which had to be more completely understood due to their short impulsive rise time characteristics, reversal of direction (in some of the force components), influence of measurement system dynamics on magnitude and time history appearance, etc.

Subsequent tests from the Ekofisk region were undertaken at the Institute for Marine Dynamics (IMD) of the National Research Council Canada, in St. Johns, Newfoundland, Canada. The test investigation took place between December 1992 and April 1993 and included tests of barrier wave-runup forces on the barrier structure and internal regions as well as the forces on the platforms (both total and deck, separately). Models of Ekofisk platforms 2/4 R and 2/4 B were tested under free field conditions and further tests were conducted with 2/4 R located in proximity to the large circular cylindrical barrier. Both regular and irregular random wave conditions were simulated in the tests, covering a number of headings for unidirectional wave systems. Vertical and horizontal plane forces were measured and Figure 5.1 illustrates the force time histories recorded.

The 2/4 B model deck measurement system only included the platform deck. For the 2/4 R model the portion of the model within the deck force measurement system also included portions of the platform vertical legs, as well as horizontal and inclined structural members. When the 2/4 R model was tested under conditions near the Ekofisk barrier structure, bridges between the structures were also included and measurements of the forces acting on the model bridges during the tests were also made.
A great effort was devoted to establishing a proper method of data analysis, since extraneous effects due to inertial reactions, etc. were present in the directly measured forces. Simple inertial corrections, in terms of the mass of the model and measurement system, would not be representative since the added mass of water associated with the model (in the vertical direction) was found to be about 15 times the model mass. In addition, the added mass was continually changing with time during the period of water contact, which further complicated any possible simple correction procedure. A number of possible data analysis and measurement correction concepts were considered and a special low-pass digital filtering method was finally adopted, with a low cut-off frequency that was less than the natural frequency of the model deck when it was partially immersed (thereby representing the complete added mass effect). This procedure was shown to account for all inertial effects, and eliminated the influence of the natural structural frequencies due to the system dynamics as well.

The 2/4 B model tested at IMD did not replicate the full scale platform deck because the second double bottom flat deck plate was omitted. On that basis direct conclusions could not be drawn for 2/4 B as impact with the lower deck plate by small wave crest heights could be significant in terms of overall structural reliability\(^{14}\). As a result a limited (unfunded) research study\(^{15}\) was carried out at IMD between October and November 1995, in order to obtain some basic test data for a double bottom model of the 2/4 B platform deck.

5.2 GULF OF MEXICO TEST PROGRAMME

As a consequence of concern about the adequacy of deck elevations particularly in the Gulf of Mexico, a model test investigation was carried out by Chevron\(^{12}\) at the Hydraulics Laboratory facility of the National Research Council Canada, in Ottawa, Canada. The study was co-supported by a number of other operating companies and has been reported most recently by Finnigan and Petrauskas\(^{33}\).

The tests were undertaken on a 1:28 scale model of a steel jacket in 141 ft water depth with a three level deck as shown in Figure 5.2.
The waves in the tests were both regular and irregular random waves, with the irregular random waves including both long-crested unidirectional as well as multidirectional conditions consistent with the Gulf of Mexico environment. The platform model included bare deck conditions, as well as conditions with different degrees of representative facility equipment mounted on the cellar deck. Tests were conducted with and without deck grating at the scaffold level. The influence of conductors on the flow was also examined. Three wave headings were modelled. A snapshot data sampling approach was adopted to avoid problems with reflected waves from the sidewalls and beach in the test tank.

The analysis of the measured test data included consideration of inertial force corrections (based on mass of the measurement system and the model segment considered) when arriving at final reported force results. The measured forces covered both the vertical and horizontal plane forces, with the major emphasis on the horizontal plane forces that were tabulated in the facility report (vertical forces were only accessible from magnetic tape records from the test facility). The test data in this study also included the forces acting on portions of the vertical platform legs, and also horizontal and inclined support members, that were included in the isolated portion of the platform model which represented the measurement region for the deck. This test data formed the basis for the recommended deck wave force prediction methods proposed by API[3].

In reducing the results, the importance of having a good estimate of the crest height became clear (see Figure 5.3) as this determines the extent of the deck affected. Furthermore for a
given trough to crest wave height, the crest portion is not uniquely defined. Imm(42) proposed an empirical method to back calculate the appropriate wave height from crest observations. This aspect is considered further in the treatment of reliability in Section 8.

![Graph 1](image1)

**Figure 5.3**

*Recorded correlation of deck force with crest height compared with wave height*(33)

The overall purpose of this section has been to outline the data against which calibration of wave-in-deck loads has been possible in order that their wider application can be considered meaningfully.

More detailed discussion of the specific calibration of the Kaplan model to the various data sets is provided in Section 6.2.

### 5.3 OTHER TEST PROGRAMMES

References to two generic test programmes have been highlighted by W S Atkins in a confidential review of wave-in-deck loading literature undertaken from a hydrodynamic modelling standpoint for Amoco(39). Both papers(50, 11) focus on the upward impact of vertical force and identify a large impulsive load preceding a smaller, slowly varying force component. It is important to recognise in the review of wave-in-deck models in Section 6 that none recognises, nor would be able to account for, this dynamic impulsive component of loading(39).
6. REVIEW OF MODELS

The survey of open literature and the various approaches made to Operators in the course of this study have revealed the following wave-in-deck load assessment models:

- Kaplan model
- Shell model
- Statoil approach
- Chevron approach
- Amoco approach.

A detailed review of each is presented in the subsections which follow. However Table 6.1 provides an initial comparison in which key features are identified. Essentially the approaches divide into two groups, namely:

- A global or silhouette approach; and
- A detailed component approach.

In general it might be considered that the first would be best suited to overall assessments of platform risks from wave-in-deck loading, particularly when detailed deck information is not available, and the second to detailed evaluation of component damage or more specific force calculations. However this categorisation should not necessarily be seen as restrictive.

It should also be noted that the methods have been presented in the literature in different contexts. For example, where a simple approach has been used to illustrate reliability implications it might be possible for the same approach to be developed to give more comprehensive information in a specific case study.

The following sections discuss the background and review the principles and calculation procedures for each model in turn. It is noted that none of the models takes dynamics into account. This may not be important globally (for a jacket) but the significance should be considered at a more local level (equipment).
<table>
<thead>
<tr>
<th>Model</th>
<th>Basis</th>
<th>Force Calculations</th>
<th>Kinematics</th>
<th>Formulae (symbols defined in review of methods)</th>
<th>Calibration</th>
<th>CoV</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Global</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Section 17</td>
<td>Drag</td>
<td>✓</td>
<td>x</td>
<td>Wave theory</td>
<td></td>
<td></td>
<td>Front face only - neglects phasing</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Wave theory</td>
<td></td>
<td></td>
<td>Simple to apply</td>
</tr>
<tr>
<td>Shell</td>
<td>Complete momentum loss on impact</td>
<td>✓    adaptation possible</td>
<td>Airy Wave</td>
<td>Airy (+ optional correction)</td>
<td></td>
<td></td>
<td>Instantaneous momentum loss should give upper bound</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Errors from wave theory assumptions</td>
</tr>
<tr>
<td>Statoii</td>
<td>Slamming</td>
<td>✓</td>
<td>neglected</td>
<td>Stokes 5th or approximation</td>
<td></td>
<td></td>
<td>Velocity distributions though wave crest considered Slamming coefficients from DNV</td>
</tr>
<tr>
<td>Component</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Kaplan</td>
<td>Inertia + impact + drag + buoyancy</td>
<td>✓    ✓</td>
<td>Wave theory</td>
<td>Wave theory (+ possible correction, eg Wheeler stretching)</td>
<td>Phillips data</td>
<td>10-15%</td>
<td>Proprietary software</td>
</tr>
<tr>
<td>Chevrem</td>
<td>Morison + drag modifications</td>
<td>✓    ✓ (only horiz addressed)</td>
<td>Stream function + wave spreading factors</td>
<td>Stream function x 1.07</td>
<td>Chevrem test data</td>
<td>33%</td>
<td>Ongoing validation against gathered data (GoM)</td>
</tr>
<tr>
<td>Amoco</td>
<td>Morison + drag/inertia modifications for slamming</td>
<td>✓    ✓</td>
<td>Wave theory</td>
<td>Wave theory + Jahnels &amp; Wheeler/ Haring &amp; Heideman</td>
<td>Kaplan model for coefficients</td>
<td>–</td>
<td>Detailed modelling</td>
</tr>
</tbody>
</table>
6.1 API RP2A SECTION 17

6.1.1 Model Description
The commentary to Section 17 of API RP2A for the Assessment of Existing Platforms presents a simple method for predicting the global wave/current forces on platform decks. The same approach has been carried through to the draft International Standard[30]. The inclusion of a wave-in-deck force calculation procedure reflects the recognition that for many older US platforms the low deck height may compromise the ability to survive extreme events. Section 17 requires that an assessment be undertaken using ultimate strength analysis if the platform has an inadequate deck height and was not designed for wave loading in the deck.

In this ‘silhouette’ approach the load is calculated based on the projected area of the deck with calibration factors to account for the density of structure and equipment. Detailed topsides information is not required, making the method general and simple to apply and thereby meeting the objectives of the API Task Group in establishing a codified approach. Inevitably such simplification of a complex problem brings some compromise and several of the organisations involved in the Task Group have developed more detailed in-house methods in parallel. Those due to Chevron, Shell and Amoco are included in the reviews below.

The API method is based on calculating the drag force from waves incident on the projected area of the deck as defined in Figure 6.1.

Specific steps in the calculation are summarised below:

- Crest height is calculated directly from the nonlinear wave theory appropriate to the location and environment (e.g. see Figure 4.1)
The silhouette area is that part of the projected area normal to the wave direction, \( \theta_w \), as shown in Figure 6.2, given by:

\[
A = A_c \cos \theta_w + A_s \sin \theta_w
\]  

(6.1)

The wetted area is the area up to the maximum crest elevation calculated for storm tide conditions.

![Figure 6.2: Wave heading and direction convention](image)

Particle kinematics from the appropriate wave theory are used to determine the maximum wave-induced horizontal fluid velocity, \( U \), in the wave crest, or at the top of the main deck silhouette if the wave is higher.

The wave/current force on the deck, \( F_{aw} \), is computed by:

\[
F_{aw} = \frac{1}{2} \rho \ C_d \left( a_{aw} U + a_{aw} U_w \right)^2 A
\]  

(6.2)

where:

- \( U_w \) is the current speed in-line with the wave
- \( a_{aw} \) is the wave kinematics factor (e.g. 0.88 for hurricanes and 1.0 for winter storms).
- \( a_{aw} \) is the current blockage factor for the jacket, and
- \( \rho \) is the density of sea water.

The drag coefficient, \( C_d \), depends on the amount of equipment on the deck as well as the wave direction.

Recommended values are given in Table 6.2 to account for different equipment densities and wave heading and are based on combined results from test data\(^\text{15}^\), theoretical considerations of momentum flux due to Mercier (see Section 6.3) and experience from hindcast evaluations following Hurricane Andrew. It should be noted that the tests and hurricane experience are related to typical Gulf of Mexico environmental conditions and recalibration for different environments may be appropriate.
Table 6.2
Drag coefficient $C_d$ for wave/current platform deck forces\(^7\)

<table>
<thead>
<tr>
<th>Deck Type</th>
<th>$C_d$ End-on &amp; Broadside</th>
<th>$C_d$ Diagonal (45°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Heavily Equipped (solid)</td>
<td>2.5</td>
<td>1.9</td>
</tr>
<tr>
<td>Moderately Equipped</td>
<td>2.0</td>
<td>1.5</td>
</tr>
<tr>
<td>Bare (no equipment)</td>
<td>1.6</td>
<td>1.2</td>
</tr>
</tbody>
</table>

- The force $F_{ak}$ is applied at an elevation $Z_{ak}$ above the bottom of the cellular deck, where $Z_{ak}$ is defined as 50% of the distance between the lowest point of the silhouette area and the lower of the wave crest or top of the main deck.

The API model only considers horizontal plane loads, which are represented by an effective drag model without explicit consideration of inertial, impact or pressure gradient effects. However, effects of currents in-line with the wave direction are included, together with other factors representing overall current blockage as well as wave kinematics factor that depends on the nature of the storm conditions. The coefficient of variation (COV) to be associated with wave-in-deck loads calculated in accordance with the API model is 35% (see Finnigan and Petrauskas\(^5\)).

6.1.2 Model Enhancement

A further validation of the API model has been undertaken at the University of California, Berkeley\(^6\) using field experience where there has been green water in the deck and platforms have survived, been damaged or failed. Publication of the Berkeley work is planned for 1998\(^5\). Preliminary results indicate that the API method has successfully predicted all failures. A new Berkeley procedure is proposed modifying the API approach in a number of ways, recognising:

- individual members rather than the silhouette area
- the run-up zone (one velocity head above the wave crest)
- the decrease in drag coefficients near the free surface.

Slamming (due to initial contact and momentum transfer) is considered in addition to inundation forces, but in general it is considered that the slamming forces are not dominant.

6.2 KAPLAN MODEL

6.2.1 Model Description

A more extensive theoretical analysis procedure for prediction of wave impact forces on platform decks is summarised by Kaplan\(^1\). While some earlier work considered a more simplified approach to this general problem (e.g. in Reference 11), and expressions provided in such cases accounted for part of the total force representation (for vertical forces), the models by Kaplan\(^1\) are more complete.

The models\(^1\) cover both vertical and horizontal forces on platform decks, which are represented essentially as horizontal flat plates with thickness. The hydrodynamic model includes inertial force effects, impact forces (due to part of the time rate of change of fluid momentum expression), and drag force contributions. The proper representation of the
added mass, both in the vertical and horizontal directions, is an important feature of the model since that term enters into the standard inertial force as well as the impact force that is proportional to the time rate of change of the added mass.

Considering the general mathematical expression for wave forces:

\[
F_i = \frac{d}{dt}[M_i V_i] + \rho S_i \frac{dV_i}{dt} + \text{drag} + \text{buoyancy}
\]

\[
= \frac{dM_i}{dt} V_i + M_i \frac{dV_i}{dt} + \rho S_i \frac{dV_i}{dt} + \text{drag} + \text{buoyancy}
\]  \hspace{1cm} (6.3)

The first term on the right hand side is the impact force term; the second term represents the inertia term, where \(M_i\) is the added mass; and the third term is due to pressure gradient effects in an unsteady flow. This third term is present on a "closed" body over which a pressure gradient acts, such as an accelerated flow with spatial dependence (as in a wave field). Initial development of the Kaplan model considered open decks\(^{18}\) and the pressure gradient was excluded. For closed structures the term is retained and a vertical buoyancy term is appropriate for structures with a double bottom.

In the work of Kaplan\(^{18}\), the various wave properties, such as the wave elevation, and kinematic terms, such as wave velocities and accelerations, are represented in a manner that is referred to as quasilinear. The crest height used is obtained either from experimental data (when comparing theory with experiment) or by use of some empirical factor relationship to the incident total wave height. That crest height, which is larger than the trough magnitude in finite water depth conditions, is a nonlinear effect although it is used as the effective amplitude of the wave elevation in the theoretical model\(^{18}\). The wave velocities and accelerations are represented in a manner similar to linear wave theory, but with the use of nonlinear Wheeler stretching relations in the vertical direction within the crest region.

Within the theoretical modelling procedure described by Kaplan\(^{18}\), some special considerations due to velocity shielding and blockage effects were applied to open deck structures wherein the deck bottom contained a relatively dense distribution of cross-members. Such considerations would not apply to closed double bottom decks, or to open deck structures with a relatively small number of cross-member elements.

A more recent extension of the work\(^{18}\) involved the use of nonlinear second order Stokes wave theory expressions for the wave elevation, velocities and accelerations applied in the general analysis procedure. The use of only second order Stokes wave theory is based on the possible extension of such a wave model to an irregular random wave representation. Some recent limited work has been carried out by Hydromechanics, Inc. via generalisation of the basic wave modelling to irregular random wave conditions, with the force representation model as given by Kaplan\(^{18}\). However almost all of the major work involved with Ekofisk field platforms as well as Gulf of Mexico representative platforms, as covered in the results\(^{18}\), has only considered regular wave conditions in the quasilinear form described above.

### 6.2.2 Correlation between Theory and Experiment

Various correlations between theory predictions and model test experimental measurements have been carried out over the past few years. Some have been presented in internal (proprietary) reports, as well as in published results\(^{19,18}\) and certain unpublished results. When considering the case of Ekofisk platforms, more extensive comparisons have been
made than for other structures. Most of the published comparisons have been made for the wave conditions where the largest forces occurred. For purposes of uniformity in considering such comparisons, as well as for application to future configurations, only the results of comparisons for free field conditions (i.e. without any barrier structure or other nearby disturbing structure present) will be discussed here.

The results given in Kaplan(18), which covered the Ekofisk platform 2/4 R and also the structure in Chevron’s investigation(12) (shown in Figure 5.2), indicated generally good correlation between theory and experiment for the larger wave conditions tested. When considering that comparisons are made between average values of both wave crest heights and maximum forces, and considering the variability in measurements of both the incident waves and the forces, the predictions are generally within about 10-15% of the measured values for solid deck plating conditions.

When porous grating is used for the upper horizontal deck surface, the vertical impact loads are reduced to about 20-25% of those for a solid deck plating (depending on the degree of porosity), where that conclusion is supported by the experimental measurements. The horizontal loads on such deck structures are essentially the same, for the same wave conditions, regardless of the type of material of the upper horizontal deck surface. This predicted result is also supported by experimental measured results.

Results for multi-deck structures, such as those considered in the tests of Chevron(12), reflected consideration of velocity blockage effects since those deck structures were made up of dense cross-members. The horizontal plane forces for such structures, without any solid equipment on the decks, were shown to be well predicted by Kaplan’s theoretical model(18). The theory was applied to the specific configurations and test conditions used by Chevron(12), with the results obtained providing support to the modelling procedure (including blockage effects) applied in that case.

Since major concern is aimed at conditions for smaller negative air gaps, some discussion of those conditions is given here. The tests at IMD on Ekofisk platforms did not cover such conditions as completely as conditions with larger waves, with larger forces, which were considered (initially, at the time of testing) as more important design conditions. In addition, for any waves with small negative air gaps, the crest peak regions would be distorted somewhat during their propagation movement along the platform since they would encounter a number of cross-members in their path. That could change the local wave shape and kinematic properties, so that conventional wave theory would not be applicable to represent the actual physical conditions in the test. Since the model of platform 2/4 B was an open model with such cross-members, and it did not have the actual double bottom plate that was present on the full scale 2/4 B platform, the information from model tests was thereby somewhat limited in utility for assessing the full applicability of the available theoretical modelling procedure.

Some measures of the validity of the theoretical methods, when compared to experimental data (in spite of these various limitations described above), were obtained by means of statistical comparison of results obtained from analysis of a large series of individual wave elevation and associated force values. These statistical measures were in the form of values of Bias and Coefficient of Variation (COV) found from establishing the ratios of measured to predicted force values. Different values were found for the horizontal plane forces (Fh, Fv) and for the vertical forces, with particular differences found for the upward vertical forces (designated as Fmax) and for the downward directed vertical forces (designated as Fmin). These statistical measures, found with the use of the recommended value C0 = 2.0 for various elements of members of both the 2/4 R and 2/4 B models are given in Table 6.3.
Table 6.3
Comparison of calculated forces with test data

<table>
<thead>
<tr>
<th>Force</th>
<th>Bias</th>
<th>COV</th>
</tr>
</thead>
<tbody>
<tr>
<td>$F_x, F_y$</td>
<td>1.00</td>
<td>0.295</td>
</tr>
<tr>
<td>$F_{max}$</td>
<td>1.24</td>
<td>0.535</td>
</tr>
<tr>
<td>$F_{min}$</td>
<td>1.04</td>
<td>0.245</td>
</tr>
</tbody>
</table>

The COV value for horizontal plane forces can be compared to that found as a result of comparisons for the API procedure. In that case, the COV value was 0.35. However the API theoretical model (given by Equation 6.2) as developed directly from the test data, was only based upon drag force considerations, and was not obtained on the basis of first principles from fundamental hydrodynamic analysis. Considering that aspect, the results from the present theoretical model procedure$^{(18)}$ can be viewed as a better procedure overall. The direct comparison shown in Table 6.3 of the procedure provides further support for the basic methodology. More recent reported results, by 11 different organizations using the API formula and their own method of modelling wave kinematics (see Reference 20), showed a COV for their calculated results to range up to 0.77 (with a lesser value of 0.45 for larger force conditions). These larger COV values do not seem to be a useful basis for significant reliance on the API method when predicting wave-in-deck forces, however, the large COV values were obtained when modelling low levels of wave inundation into the deck and hence the apparently high variability may not be a true reflection of the applicability or otherwise of the procedure itself given the sensitivity to the wave / crest height determined.

6.2.3 Modified Procedure and Results for Double-bottom Structure
As mentioned earlier, test data for a double bottom model of the Ekofisk 2/4 B model have recently been obtained$^{(19)}$. A modified theoretical procedure, primarily based on the methods of Kaplan$^{(18)}$ and described in general by Equation 6.3, has been applied to that configuration and comparisons made between theory and experiment. While those results, both for the experiments and the theoretical procedure, are proprietary (to IMD and Hydromechanics, Inc.), some of the results of the comparisons (which are still not fully completed, to date) can be described.

The model tests only considered waves that were in the longitudinal direction ($\alpha = 0^\circ$, in the notation of Kaplan$^{(18)}$), and the data analysis methods were the same as described by Murray et al.$^{(19)}$. Final results of these tests were given as average values for a group of eight oscillation cycles, and comparisons were made on that basis. As a consequence of the unpublished nature of these results, no detailed “picture” of the force time histories (and possible variability in the sequence of the oscillations) was available for visual examination.

The calculation method was essentially the same as in previous cases for determining the vertical force (both the positive maximum upward force, and the negative downward force), but with inclusion of the buoyancy force and the vertical pressure gradient term. The calculations for the wave forces have been carried out using both the quasilinear wave models described above (that have been used in prior work, as described in the work by Kaplan$^{(18)}$), as well as the nonlinear second order Stokes wave representations.

For the determination of horizontal plane longitudinal forces, some changes in the computation procedure have been developed for the closed body case, which are still similar to those used for the model test cases treated previously. The impact force term is
essentially localised at the leading edge region where the wave contacts the deck, while the inertial force term, the pressure gradient term, and the drag force term are found in a continuous manner that accounts for the spatial variability of the wave progressing along the length of the deck. The earlier horizontal plane force calculations considered forces that were effectively localised at the various cross-members along the length of the deck, using wave property values evaluated at the centre of each element as the wave progresses. The treatment for the closed body is essentially equivalent to that procedure. The major difference is the need for separate values of the drag coefficient for horizontal flow and for vertical crossflow, since the effect of the relative proportions of the deck height and the deck length result in a different (lower) value of drag coefficient in the horizontal direction.

The comparison between theory and experiment for the case of the 2/4 B Ekofisk platform, which is generally similar to other platforms that have tank bottoms, etc., seems to support the use of the quasilinear wave field representations rather than the nonlinear second order model (for the same crest height). The nonlinear wave model results are generally larger than those found by the quasilinear wave model, with very much larger negative vertical wave forces for the nonlinear predictions that exceed both the quasilinear predicted values as well as the measured values.

While the positive vertical force values are generally similar for both the quasilinear and nonlinear model results, the horizontal forces from nonlinear wave theory are always larger than those from the quasilinear model. The horizontal forces from the quasilinear model, when compared to the recent IMD test data, show values that are both larger and smaller than the test data, depending upon the particular wave period and the initial calm water clearance height (i.e. the air gap without waves). These differences range from 5-20%, except for the smallest values (for the smallest crest heights) where a difference of about 1.0 MN would represent a percentage deviation from the measured value of about 80%. In view of these small differences, and without any very precise measure of possible experimental test measurement errors (in both the wave and force measurements), the comparison here generally supports the use of the Kaplan theoretical model. This model includes the quasilinear wave representation, use of different drag coefficient values, and the computation procedure that considers the spatial variability of force components as the wave progresses along the deck.

A limited comparison of the results of using the simple API formulation for predicting the longitudinal forces for the 2/4 B closed platform shows that there are some particular cases where that procedure can show agreement with test data, as well as cases where the agreement is not that good. Considering the basic data set from which the API formulae arose, actual measured force values showed a difference of about 15% for conditions with a full equipment deck as compared to a bare deck. The API formula would indicate an increase of 50% or more (depending on the wave elevation and the selected silhouette area) for such cases, which is not a reasonable estimate. Thus there is some basis to question the applicability of that general procedure for arbitrary configurations and conditions. In addition, the API method does not treat vertical wave impact forces on deck structures at all.

6.3 SHELL'S WAVE-IN-DECK MODEL

6.3.1 Introduction
Mercier developed a method for estimating wave loads on decks based on conservation of momentum principles. The paper in which this method is reported remains confidential to Shell Oil, and therefore is not accessible to this study. The approach proposed by van
de Graaf, Tromans and Vanderschuren\textsuperscript{22} for assessing wave-in-deck loads derives from the work of Mercier and has been made available for this HSE study with the kind permission and cooperation of Shell UK Exploration and Production and KSEPL in the Netherlands. The basis and application of the method is also described in Reference 26. The method has been implemented in spreadsheet format, a copy of which has been made available for this study and is used as a basis for comparison in Section 7.

Discussion with Dr Mercier\textsuperscript{29} confirmed that his method was more rigorous than either the API or KSEPL methods\textsuperscript{22}, however, as his original model involves triple integrals and control volumes, it is rather specialised and not easy to apply routinely. Discussion with Mercier\textsuperscript{29} confirmed his support for the KSEPL method for practical application.

The simplification in the KSEPL approach is the assumption that the top of the wave which hits the deck is 'shaved off' during the passage of the wave and the forward horizontal momentum of the water particles in that portion of the wave which hits the front and bottom of the deck is completely and instantly destroyed providing an upper bound solution. This requires the bottom of the deck to be transparent, (i.e. consist of open deck grating) such that the wave can enter the deck from below and once in the deck, the horizontal water particle motion is then stopped by the vertical areas of the deck support beams and facilities such as piping, conductors, wellheads, pressure vessels etc.

The linear Airy Wave Theory is used to calculate the water particle kinematics, providing a physically sound and practical solution for deep water applications. The inaccuracies in the model are primarily related to the prediction of horizontal crest velocities and crest height on this basis but the authors\textsuperscript{22} suggest that to achieve better accuracy, the Airy wave amplitude and horizontal water particle velocities could be adjusted or calibrated to match better estimates using more accurate wave theories.

### 6.3.2 Basic Theory

Figure 6.3 illustrates the assumed interaction of the wave with the deck as it passes through the structure. Lateral loading is generated on the deck face and on vertical faces behind, which are reached by waves passing up through the underside of the deck. The force components clearly depend on the phase of the wave with respect to the structure.

![Figure 6.3: Wave on front and bottom of deck](image_url)
Distinct behaviours can be identified, when:

- The wave crest enters and passes the front of the deck \((t_1, t_2)\);
- The wave crest enters and passed the rear of the deck \((t_4, t_5)\); and
- The crest maximum passes the front and rear of the deck \((t_3, t_6)\).

Based on the Airy Wave theory, the generated equation for the wave crest hitting the deck is:

\[
\zeta = a \cos \left( \frac{2\pi(L/2)}{\lambda} - \frac{2\pi t}{T} \right) \times h_d
\]  

(6.4)

where

- \(\zeta\) = wave elevation as a function of time and space
- \(a\) = wave amplitude
- \(\lambda\) = wave length
- \(T\) = wave period
- \(L\) = length of deck
- \(h_d\) = height of deck above sea level.

The times, relative to \(t = 0\) and \(x = 0\) in Figure 6.3, defining the different stages, are:

\[
t_1, t_3 = -\left\lfloor \frac{L}{2\lambda} \pm \frac{\alpha}{2\pi} \right\rfloor \quad \text{where} \quad \alpha = \arccos \left( \frac{h_d}{a} \right)
\]

\[
t_4, t_5 = -\left\lfloor \frac{-L}{2\lambda} \pm \frac{\alpha}{2\pi} \right\rfloor
\]

\[
t_2, t_6 = \pm \left\lceil \frac{L}{2\lambda} \right\rceil
\]

At each instant in time between \(t_1\) and \(t_4\), the wave crest hitting the deck can be considered as a quasi-stationary process and the expression for the momentum balance applied:

\[
F = \dot{m}v
\]  

(6.5)

where

- \(\dot{m}\) = rate of mass flow
- \(v\) = initial horizontal velocity

The force components on the front and bottom of the deck are calculated independently.

**6.3.2.1 Force on front of deck**

The force on the front of the deck occurs between times \(t_1\) and \(t_2\). The mass flow through the control surface of the deck front during this time interval is:
\[ m = \rho b (\zeta - h_d) (u + u_c) \]  \hspace{1cm} (6.6)

where \( \rho \) = density of water  
\( b \) = width of deck  
\( u \) = horizontal wave particle velocity  
\( u_c \) = current velocity.

Assuming that the gross forward momentum passing through the front of the deck is completely and instantaneously dissipated, the force is obtained from the equation for momentum balance:

\[ F_f(t) = \rho b (\zeta - h_d) (u + u_c)^2 \]  \hspace{1cm} (6.7)

From the Airy Wave Theory, the amplitude of the water particle velocity at the water surface is:

\[ u = K_e \omega a \cos \left( \frac{2\pi(L/2)}{\lambda} - \frac{2\pi t}{T} \right) \]  \hspace{1cm} (6.8)

where \( \omega = \frac{2\pi}{T} \)  
\( K_e \) = kinematics enhancement factor by way of connection to the Airy modelling assumptions.

The force on the front deck is therefore:

\[ F_f(t) = \rho b \left[ K_e \omega a \cos \left( \frac{2\pi(L/2)}{\lambda} - \frac{2\pi t}{T} \right) + u_c \right]^2 \cos \left( \frac{2\pi(L/2)}{\lambda} - \frac{2\pi t}{T} \right) - h_d \]  \hspace{1cm} (6.9)

where it can be seen that the \( K_e \) factor modifies the velocities (i.e. in the first term) but no asymmetry is allowed for in the wetted deck area calculations (second term).

6.3.2.2 Lateral force due to water passage through underside of deck

Lateral forces on the underside of the deck occur between times \( t_i \) and \( t_k \). The mass flow passing through an element of length \( dx \) of the control surface of the deck bottom at any time during the time interval \( t_i \) to \( t_k \) is:

\[ dm = \rho b v dx \]  \hspace{1cm} (6.10)

where \( v \) = vertical water particle velocity in the crest.

Similarly to the force on the front of the deck, it is assumed that the gross forward momentum passing through the element \( dx \) is completely and instantaneously dissipated. The force on the element \( dx \) is obtained from the equation for momentum balance as:

\[ dF = dm u = \rho b v dx u \]  \hspace{1cm} (6.11)

The total force on the bottom of the deck is obtained by integration:
\[ F = \int_{x_c}^{x_f} \rho b \nu u \, dx \]  (6.12)

where \( x_f \) = location along the bottom of the deck of the front of the wave or, when the front of the wave has passed the back of the deck, the location of the back of the deck

\( x_c \) = location of the front of the deck or, when the crest has passed the front of the deck, the location of the wave crest along the bottom of the deck.

After the crest has passed a specific location along the deck bottom, the vertical water particle motion would be downwards and there is no longer any influx into the bottom of the deck at this location.

Using Airy Wave Theory the horizontal and vertical water particle velocities in the crest are given by:

\[ u = K_a \omega a \cos \left[ \frac{2\pi(x_f)}{\lambda} - \frac{2\pi}{T} \right] \]

\[ v = \omega a \sin \left[ \frac{2\pi(x_f/2)}{\lambda} - \frac{2\pi}{T} \right] \]  (6.13)

The force on the bottom of the deck as a function of time between \( t_i \) and \( t_f \) is obtained by integration giving:

\[ F_b(t) = \rho b K_a \omega^2 a^2 \frac{\lambda}{8\pi} \]  (6.14)

\[ \cdot \left[ \cos \left( \frac{4\pi x_f}{\lambda} - \frac{4\pi t}{T} \right) - \cos \left( \frac{4\pi x_c}{\lambda} - \frac{4\pi t}{T} \right) \right] + \frac{4\nu}{K_a \alpha} \left[ \cos \left( \frac{2\pi x_f}{\lambda} - \frac{2\pi t}{T} \right) - \cos \left( \frac{2\pi x_c}{\lambda} - \frac{2\pi t}{T} \right) \right] \]

6.3.2.3 Total lateral force

The total lateral force on the deck is the sum of the two components:

\[ F_{\text{en}}(t) = F_i(t) + F_b(t) \]

Van de Graaf et al.\(^{22}\) report wave-in-deck force calculations for the 225ft West Delta 103A platform in the Gulf of Mexico, which had been analysed previously by Mercier\(^{21}\) following a calibration of his model against test data.

Table 6.4 provides the comparison of the analytical predictions, giving the best correlation when only the Airy crest height (and not horizontal particle velocity) in the Van de Graaf model was adjusted to fit the Chappell wave accounted for by Mercier. In these cases representing different wave/crest heights and degrees of inundation, Mercier had assumed
practically all momentum is dissipated in the deck. As a result it could be concluded that
the remaining differences are attributable to the use of Airy theory in the calculation of wave
kinematics and profile.

The best results were obtained by adjusting only the Airy wave crest to fit the Chappelar
crest. This is done such that the Airy wave height is twice the Chappelar wave crest. The
difference in results may be attributed to the use of Airy wave theory to calculate the wave
kinematics and profile.

<table>
<thead>
<tr>
<th>Case</th>
<th>Airy crest adjusted to fit Chappelar crest</th>
<th>Airy crest and hor. velocity adjusted to fit Chappelar crest</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case I</td>
<td>-9%</td>
<td>+14%</td>
</tr>
<tr>
<td>Case II</td>
<td>+1%</td>
<td>+24%</td>
</tr>
</tbody>
</table>

6.3.2.4 Summary
The authors' state that considering all the uncertainties involved in calculating extreme
crests and the corresponding kinematics, the calculation of deck loads using this model
provides a simple but effective solution of sufficient accuracy.

The method is particularly applicable for decks which allow the wave to enter from below,
and block the wave's horizontal motion by the presence of facilities such as pressure vessels
etc. For platforms where there is little or no obstruction and the wave can wash through the
deck, this approach may be too conservative.

For decks which are not transparent for the vertical wave motion, blocking the wave to enter
from below, the horizontal deck load will consist only of the component on the front of the
deck. However, there will be a vertical wave load acting upwards on the bottom of the
deck. This vertical wave load can also be simply calculated using an equation similar to that
for horizontal load on the deck bottom, but with the horizontal velocity replaced by the
vertical velocity. Since the vertical velocities in a crest are small, this load is considered
by the authors\(^\text{22}\) to be relatively small when compared to the horizontal load on the bottom
of a similar 'transparent' deck. However, it should be recognised that initial slamming
effects from the upward motion are followed a second or two later by downward inertia
effects which can be significant due to the large vertical accelerations in the crest; these
cannot be captured by the Shell model.

6.4 STATOIL APPROACH

The Statoil approach was covered briefly in two studies of system reliability\(^\text{17, 27}\). The
description available indicates that only the wave impact pressure force (or slamming) on
components is considered, given by:

\[
F_{eq} = P_{eq} A_c = \frac{1}{2} \rho c v^2 \left( c - h_d \right) \text{ for } c > h_d
\]

\( (6.15) \)
where \( P_{eq} \) = impact pressure force
\( A_x \) = deck area exposed to wave impact
\( u \) = normal water particle velocity
\( c \) = crest height above still water level (SWL)
\( h_d \) = deck height above SWL
\( b \) = deck width
\( \rho \) = seawater density
\( C_{slam} \) = slamming coefficient.

The model may be considered as a simplification of the Kaplan model (Section 6.2) omitting secondary terms, or a variant of the Shell approach (Section 6.3) with the slamming coefficient determining the degree of momentum change assumed. Dalane and Haver\(^{(27)}\) adopted a slamming coefficient of 3.0, based on DNV guidelines for horizontal members in a wave impact zone, in conjunction with water particle velocities at a representative height with respect to the exposed deck area. Haver\(^{(17)}\) adopted a similar approach but combined a slamming factor of 2 with particle speeds at the wave crest.

The crest height and particle velocities, whether at or through the crest, are derived using Stokes 5th Order wave theory with approximate relations given for particle velocities\(^{(27)}\):

\[
 u (c,z) = (6.3 + 0.1014z^{1.17}) (0.4 + 0.0375c) \quad (6.16)
\]

and crest height (for 70-75m water depths)\(^{(17)}\):

\[
 c = 0.36H^{1.16} \quad (6.17)
\]

Specific descriptions of calibrations are not provided but the associated publications use the simple model to demonstrate the sensitivity of reliability predictions to the loading and resistance parameters and to illustrate simply the potential significance of the wave-in-deck loading phenomenon.

### 6.5 CHEVRON APPROACH

As described in Section 5, Chevron Petroleum Technology Company has been at the forefront of experimental investigations to evaluate the consequences of waves encroaching platform decks. The results formed the basis for the silhouette method in RP2A Section 17\(^{(7)}\) developed by the API Task Group in which Chevron participated.

In a recent publication\(^{(23)}\), Chevron recommend the method as a generalised procedure for wave-in-deck load calculations, particularly in instances where incomplete detail of the structure and exposed equipment is available.

However for more accurate evaluations, an alternative component based approach is proposed utilising a numerical model of the deck to calculate the local loads as the wave passes through the structure. The wave forces are computed using Morison's formulation with specific drag coefficients depending on wave approach direction, equipment density and conductors and an inertia coefficient of 1.5. On the basis that horizontal crest accelerations are near zero and crest velocities dominate when crests are high enough to impact the deck, it is considered that the deck forces are insensitive to inertia effects.

Specific practices for implementing the approach are as follows:
• The computer model includes deck legs; vertical, horizontal and diagonal support tubulars; plate girders, truss beams, deck beams, deck grating and deck equipment. Based on test data calibrations it is recommended that grating be modelled as lumped areas through the deck using only 35% of the actual projected area in the lateral direction. Deck equipment is characterised by rectangular and circular cylinders.

• Drag coefficients are selected from Table 6.5 which it should be noted relate only to open sided deck structures. The values are reported to be "based on a rudimentary blockage procedure in which flow is assumed to be steady relative to the duration of wave impact".

<table>
<thead>
<tr>
<th>Table 6.5</th>
<th>Calibrated coefficients from Chevron component based wave-in-deck model(^{(33)})</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Heading</strong></td>
<td><strong>With equipment</strong></td>
</tr>
<tr>
<td></td>
<td>Without conductors</td>
</tr>
<tr>
<td>end-on</td>
<td>0.60</td>
</tr>
<tr>
<td>diagonal</td>
<td>1.00</td>
</tr>
<tr>
<td>broadside</td>
<td>0.85</td>
</tr>
</tbody>
</table>

• Wave kinematics are based on Stream Function methods with a 7% increase in crest height on the basis of Chevron’s in-house comparison of stream function predictions with measured crest heights (see also Section 4.4).

• Directional spreading effects are accounted for with factors of 0.88 and 0.95 applied to fluid velocities for hurricane and winter storm conditions, respectively, again based on experimental calibrations reflecting Gulf of Mexico conditions.

• Wave forces are calculated for a range of wave heights for the peak spectral period and plotted to describe the increase in deck load with increasing crest height. For any specific crest height the corresponding force can be taken appropriately from the graph, with due regard to any sensitivity to small changes in crest height calculation.

Calibration of the component based approach was initially undertaken with respect to the 1988/89 programme of tests (Section 5.2) but subsequently Chevron has maintained a database of measured crest and wave height data against which the mathematical models have been evaluated. The approaches described in this section were published by Chevron in 1997 and may therefore be considered to represent their state-of-the-art experience.

However, it is to be recognised that the focus of the developments has largely been in the Gulf of Mexico for which multidirectional waves more accurately reflect the steep design waves than regular unidirectional waves which are relatively unstable and develop unsteady force statistics in the wave basin.

Validation of the component model under these conditions is shown in Figure 6.4 where the calculated increase in deck force with greater penetration of the crest on successive deck
levels is compared with test data. The overall comparison of test results and predictions for 532 conditions is shown in Figure 6.5. A bias of just 0.5% is calculated but with a coefficient of variation of 33%.

![Graph showing comparison of measured and predicted deck forces for moderately equipped deck, broadside loading.](image1)

**Figure 6.4**

Chevron approach - comparison of measured and predicted deck forces for moderately equipped deck, broadside loading

![Graph showing measured versus predicted deck forces for all 532 impact data points.](image2)

**Figure 6.5**

Chevron approach - comparison of measured and predicted deck forces for all 532 impact data points

### 6.6 AMOCO APPROACH

The basis of the Amoco approach is to adopt Morison's equation (Equation 4.1) and modify the coefficients to give an effective representation of wave-in-deck loading phenomena which Kaplan\(^{(10)}\) characterised in terms of drag, inertia, buoyancy and slamming (Equation 6.2). Buoyancy is neglected in the Amoco work but the drag and inertia coefficients are
modified to capture the high initial impacts through calibration to the more complex Kaplan equations derived from technical data.

Initially Amoco considered only horizontal impacts on horizontal cylindrical members but more recently this work has been extended to encompass vertical loads and inclined and vertical cylinders. Calibration to Kaplan's method was achieved using curve fitting techniques and vectorial residual error methods. Results of that work have been made available to the study by the kind permission of Amoco Worldwide Engineering and Construction.

Table 6.6 summarises the equivalent drag and inertia coefficients which result, together with the comparison of predictions. A similar approach has been adopted by other authors in the past and Table 6.7 compares the recommendations for horizontal cylinders to which the studies were restricted.

<table>
<thead>
<tr>
<th>Force direction</th>
<th>Authors</th>
<th>$C_d$</th>
<th>$C_m$</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Horizontal</td>
<td>Davies and Martin, 1990</td>
<td>0</td>
<td>2.5</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Imm, 1991</td>
<td>(1.0?)</td>
<td>5.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Gebara, 1996</td>
<td>1.0</td>
<td>3.3</td>
<td></td>
</tr>
<tr>
<td>Vertical</td>
<td>Davies and Martin, 1990</td>
<td>1.0</td>
<td>1.5</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Imm, 1991</td>
<td>(1.0?)</td>
<td>1.2</td>
<td>Model scale</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>(1.0?)</td>
<td>0.49</td>
</tr>
<tr>
<td></td>
<td>Gebara, 1996</td>
<td>3.9</td>
<td>2.0</td>
<td></td>
</tr>
</tbody>
</table>

In calculating wave-in-deck forces, the loading on cylinders is based on the above and detailed recommendations are presented for plate / grating and for underdeck stringers accounting for effects of shielding.

Wave kinematics in general are based on Airy wave theory with an expression for the crest elevation ($\eta_c$) for large nonlinear waves given by:

$$\eta_c = \frac{H}{2} \exp\left(\frac{H-k}{2}\right)$$  

(6.16)

where $H$ = wave height, and $k = \text{wave number} = 2\pi/L$ with $L = \text{wave length}$.

Consideration is given to the modifying influence of wave-deck interactions and a recommendation is given that wave crest elevations should be calculated using the Jahns and Wheeler procedure with coefficients from the work of Haring and Heideman.

Comparisons between Amoco's approach and the Section 17 method were presented in Reference 39 and these are discussed in Section 7.
### Table 6.6
Equivalent Morison's Equation Cd and Cm cd comparison with Kaplan predictions for large diameter elements (D>16")

<table>
<thead>
<tr>
<th>Member inclination</th>
<th>Wave direction w.r.t member</th>
<th>Force component</th>
<th>General - all components</th>
<th>Initial impact - applicable to one truss only at any time</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Cd</td>
<td>Cm</td>
</tr>
<tr>
<td>Horizontal</td>
<td>Normal</td>
<td>Horizontal</td>
<td>1.0</td>
<td>2.0</td>
</tr>
<tr>
<td>Horizontal</td>
<td>Normal</td>
<td>Vertical</td>
<td>1.0</td>
<td>2.0</td>
</tr>
<tr>
<td>Horizontal</td>
<td>Parallel</td>
<td>Vertical</td>
<td>1.0</td>
<td>2.0</td>
</tr>
<tr>
<td>Vertical</td>
<td>All</td>
<td>Horizontal</td>
<td>1.0</td>
<td>2.0</td>
</tr>
<tr>
<td>Inclined</td>
<td>Normal</td>
<td>Horizontal</td>
<td>1.0</td>
<td>2.0</td>
</tr>
<tr>
<td>Inclined</td>
<td>Normal</td>
<td>Vertical</td>
<td>1.0</td>
<td>2.0</td>
</tr>
<tr>
<td>Inclined</td>
<td>Parallel</td>
<td>Horizontal</td>
<td>1.0</td>
<td>2.0</td>
</tr>
<tr>
<td>Inclined</td>
<td>Parallel</td>
<td>Vertical</td>
<td>1.0</td>
<td>2.0</td>
</tr>
</tbody>
</table>

*Note: buoyancy effects require separate consideration*
6.7 SUMMARY

Based on the detailed appreciation of the alternatives for calculating wave-in-deck loads above, the summary presented in Table 6.1 can be revisited and the following observations made:

- All but the Shell model rely on empirical coefficients (e.g. $C_d$ or $C_m$). For the Statoil model these are accepted coefficients for wave loading on members. In the other cases coefficients are derived from sources of wave-in-deck load test data. The applicability of the type of seas and form of deck structures in the tests to any specific installation need to be considered. Some adjustment to coefficients may be appropriate.

- Consideration of vertical forces is inconsistent. Where the ‘silhouette’ methods neglect these effects, vertical load contributions are an important part of the Kaplan approach, for example. The difference may be due to the type of structures and degree of inundation anticipated. For a structure with a platted base deck below the beams, significant vertical loads may be anticipated even for limited air gap exceedance. With uncased underdeck beams the relative effect would be much less, consistent with the silhouette approaches, however, there would be a cumulative lateral load on underdeck beams as the wave passes through the structure. The applicability of the deck construction assumptions to any specific application need to be carefully reviewed in selecting an appropriate model.

- The engineering effort to implement the various models varies significantly. (The comparative accuracy is examined in Section 7.) The global silhouette models only require overall structural parameters and approximations of equipment densities and deck transparency. The component models require individual equipment and structural elements to be modelled and in some cases this extends to determining effective profiles and selecting appropriate drag and inertia coefficients. As a consequence the computational effort is greater. From the comparison of the Section 17 and Chevron models in Table 6.1, it would appear that the uncertainty is no less.

- The effective interpretation of the physics varies between the models. Finnigan and Petrauskas[30] argue that the resultant loads are insensitive to inertia on account of the near zero horizontal accelerations in the wave crest. Explicit and effective inertial effects vary more significantly in the Kaplan and Amoco approaches. The impact of the wave crest on the deck is interpreted in terms of drag (Section 17, Chevron) or slamming (Statoil, Kaplan).

- In general the methods adopt wave theories to capture the kinematics with different adjustments being introduced to characterise wave crest heights and so reflect calculations of the affected areas. The more sophisticated are probably those of Jahns and Wheeler[30] and Haring and Heidema[31] but Chevron’s database from which a 7% increment on the stream function crest has been determined provides an important reference base.

- Little reference is made to the effective elevation of the lateral force resulting from the wave-in-deck inundation and, apart from the Statoil model, it is not apparent that the variation of particle velocities away from the crest itself is addressed specifically. For certain modes of system collapse the moment arm
may be significant, demanding special attention to the anchoring of equipment, be it to main beams above or below the wetted area.

Beyond these initial observations, the comparisons for selected methods in Section 7 provide further insight.
7. QUANTIFIED COMPARISON OF THE VARIOUS WAVE-IN-DECK LOAD MODELS

The purpose in providing a quantitative comparison of the wave-in-deck loading models presented in Section 6 is firstly:

- To indicate the significance of different modelling assumptions on the calculated loads,

and secondly

- To provide a basis for recommending a practical and meaningful approach to account for wave-in-deck loads in structural reliability evaluations for UK sector installations.

7.1 MODEL SELECTION

With that premise it is sensible for attention to focus on the simpler global silhouette models rather than the more detailed component evaluations. Table 7.1 gives a high level screening of the models for the purposes of the present study.

<table>
<thead>
<tr>
<th>Method</th>
<th>Advantages</th>
<th>Disadvantages</th>
</tr>
</thead>
<tbody>
<tr>
<td>Section 17</td>
<td>• Straightforward to combine with jacket analysis</td>
<td>• GoM calibration of coefficients?</td>
</tr>
<tr>
<td></td>
<td>• Backed by industry consensus</td>
<td></td>
</tr>
<tr>
<td>Shell</td>
<td>• Independent of environmental calibrations</td>
<td>• Upper bound</td>
</tr>
<tr>
<td></td>
<td>• Programmable</td>
<td></td>
</tr>
<tr>
<td>Statoil</td>
<td>• Simple to apply</td>
<td>• Calibration undefined adopted for illustrative purposes</td>
</tr>
<tr>
<td>Kaplan</td>
<td>• North Sea application</td>
<td>• Proprietary software (but commercially available)</td>
</tr>
<tr>
<td></td>
<td>• Relatively widely used</td>
<td></td>
</tr>
<tr>
<td></td>
<td>• Explicit account of vertical loads (+ and -)</td>
<td></td>
</tr>
<tr>
<td>Chevron</td>
<td></td>
<td>• Detailed deck model required</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• GoM based $C_a$ coefficients</td>
</tr>
<tr>
<td>Amoco</td>
<td></td>
<td>• Detailed deck model required</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Detailed assignment of $C_m$, $C_l$ coefficients</td>
</tr>
</tbody>
</table>
It should be emphasised that the advantages and disadvantages might reverse for different applications and Table 7.1 should not be read as a definitive evaluation. For examining local damage or the vulnerability of equipment, the first approaches would be inadequate and it would be essential to use more detailed evaluations which the Kaplan, Amoco and Chevron methods offers. These approaches would also be well suited to more critical safety evaluations where a deck model is available.

However for the purposes of assessing the significance of wave-in-deck loads in terms of UK sector platform reliability in the course of platform pushover analyses, it is considered that the first four models are more appropriate. Comparative load predictions for a sample deck in a Northern North Sea environment is confined to these models and results are presented in Section 7.3.

As background to the specific evaluations, potential sources of comparison were examined in the open literature. In all but one case, a single model was adopted throughout. The exception was associated with the Amoco developments where comparison was made between the detailed component based method and the API RP2A Section 17 simplified silhouette approach. Those results are reproduced here in Section 7.2 as they enable some evaluation of the detailed methods to be retained whilst providing insight to the most widely available Section 17 approach.

### 7.2 Amoco - Section 17 Comparison

An assessment of the wave-in-deck loads is presented in Reference 39 for the WD90B Platform in the Gulf of Mexico. Details of the geometry are not given but different degrees of congestion are postulated for ‘moderately’ and ‘heavily’ equipped configurations. Fifty and 100-year conditions are considered for three platform headings. In applying Section 17 the wave kinematics were calculated using either the recommended wave theory (10th order stream function) or incorporating a Jahn and Wheeler distribution. The Amoco modified Morison procedure calculations used Kaplan’s wave/crest modelling approach based on Airy theory.

Key comparisons are presented in Table 7.2 where $F_x$ and $F_y$ are forces in the horizontal plane and $F_z$ acts vertically. On account of the base formulation (see Section 6.1) the vertical load components are zero (neglected) in the Section 17 calculations. $F_x = 0$ represents a no contact condition.
Table 7.2
Representative comparisons of Amoco and Section 17 wave-in-deck loads\(^{(38)}\)

<table>
<thead>
<tr>
<th>Model</th>
<th>End-On 100 year</th>
<th>50 year</th>
<th>Diagonal - 28.25 100 year</th>
<th>50 year</th>
<th>Diagonal - 53.5 100 year</th>
<th>50 year</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>(F_x = 741.7)</td>
<td>(P_x = 222.8)</td>
<td>(F_x = 473.0)</td>
<td>(F_x = 141.7)</td>
<td>(F_x = 113.5)</td>
<td>(F_x = 0.0)</td>
</tr>
<tr>
<td></td>
<td>(F_y = 317.7)</td>
<td>(F_y = 95.2)</td>
<td>(F_y = 19.1)</td>
<td>(F_y = 0.0)</td>
<td>(F_y = 0.0)</td>
<td>(F_y = 0.0)</td>
</tr>
<tr>
<td>2</td>
<td>(F_x = 298.2)</td>
<td>(F_x = 0.0)</td>
<td>(F_x = 189.7)</td>
<td>(F_x = 0.0)</td>
<td>(F_x = 0.0)</td>
<td>(F_x = 0.0)</td>
</tr>
<tr>
<td></td>
<td>(F_y = 127.4)</td>
<td>(F_y = 0.0)</td>
<td>(F_y = 0.0)</td>
<td>(F_y = 0.0)</td>
<td>(F_y = 0.0)</td>
<td>(F_y = 0.0)</td>
</tr>
<tr>
<td>3</td>
<td>(F_x = 701.4)</td>
<td>not analysed</td>
<td>(F_x = 553.0)</td>
<td>not analysed</td>
<td>(F_x = 213.1)</td>
<td>not analysed</td>
</tr>
<tr>
<td></td>
<td>(F_y = 987.0)</td>
<td></td>
<td>(F_y = 159.6)</td>
<td></td>
<td>(F_y = 490.1)</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>(F_y = 948.7)</td>
<td></td>
<td>(F_y = 22.4)</td>
<td></td>
</tr>
</tbody>
</table>

Key:
Model 1: Section 17: Crest elevation as per Jahn and Wheeler
Model 2: Section 17: Crest elevation as per stream function wave theory and storm tide
Model 3: Storm tide: Amoco modified Morison procedure

A number of observations can be made as follows:

- The modified Morison procedure indicates vertical loads can be significant but these are neglected in the Section 17 model.

- Basing crest height on the wave theory significantly underestimates the deck loading compared with the modified Morison procedure results. It can be seen that the potential for inundation is not identified in some cases, however this indicates disparity in the crest calculation methods rather than the loading calculations.

- With comparable sophistication in crest height calculations the first and last rows indicate comparable magnitudes of resultant lateral force for two of the three directions. In the third case sources of divergence are the low level of inundation and consequent sensitivity of force to equipment density (assumed uniform in Section 17) and to contributions from underdeck members behind the front face.

Further observations\(^{(39)}\) were that:

- The modified Morison results presented the maximum force combination aligned with the wave heading for any step through the cycle whereas Section 17 delivers a single finite value.

- The phase considerations within Amoco's modified Morison approach enable the maximum combination of jacket and deck loads to be considered which may be less onerous than the individual maxima necessarily adopted when using Section 17.

- The modified Morison approach showed that for substantial inundation, 40-60% of the load acts at the bottom deck level such that the Section 17 recommendation to site the momentum at half the inundation depth could be over conservative.
Overall the authors concluded that Amoco's modified Morison's approach was more rigorous than Section 17. Nevertheless, for significant inundation, a reasonable measure of lateral forces could be obtained simply using Section 17. Particular caution is recommended for low levels of inundation as the neglect of underdeck loads may be unconservative in terms of the total net force.

These are important findings to carry forward to the specific model evaluations in the following section.

7.3 WAVE-IN-DECK LOAD CALCULATIONS

For the purpose of these example calculations, a Northern North Sea structure is considered with an underdeck geometry represented by the beam grillage in Figure 7.1. A range of forms of deck construction is examined from full plating to grating with an effective porosity of 80%. In addition the potential for the underside of the beams to be plated to create a tank deck is considered such that lateral loading is incident only on the outermost deck beams.

The deck above the beams is considered either to be open or fully clad with a vertical wall around the deck perimeter above the beams. The configurations are illustrated in Figure 7.1. Various hydrodynamic parameters are given in Table 7.3 where the base case figures are considered to be representative of Northern North Sea locations and the associated range provides a plausible basis for practical variations.

Table 7.3
Wave-in-deck load calculation parameters

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Base case</th>
<th>Range</th>
<th>Units / Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Storm still water level including tide and surge</td>
<td>184.830</td>
<td>-</td>
<td>Metres</td>
</tr>
<tr>
<td>Wave height (H)</td>
<td>29.250</td>
<td>22.58 - 29.25</td>
<td>Metres</td>
</tr>
<tr>
<td>Wave period</td>
<td>12.800</td>
<td></td>
<td>Seconds</td>
</tr>
<tr>
<td>Current velocity</td>
<td>0.715</td>
<td></td>
<td>m/s</td>
</tr>
<tr>
<td>Crest ratio, \eta/H</td>
<td>0.600</td>
<td>0.55-0.65</td>
<td>-</td>
</tr>
<tr>
<td>Elevation of deck underside above storm still water</td>
<td>13.550</td>
<td>9.55-17.55</td>
<td>m</td>
</tr>
<tr>
<td>Wave heading</td>
<td>-</td>
<td>90°, 45°, 0°</td>
<td>Broadside, diagonal, end-on</td>
</tr>
<tr>
<td>Solidity</td>
<td>100%</td>
<td>20-100%</td>
<td>Solidity = 1-porosity 100% = full deck plating 20% = grating with 80% openings</td>
</tr>
</tbody>
</table>
Figure 7.1
Sample platform deck layout
For the purposes of illustrating the wave in deck force calculation procedures, the base case corresponds to a finite degree of inundation. However reference to design conditions and the HSE Guidance requirement for airgap

The base case environmental conditions in Table 7.3 relate to a 100-year return period at the specific location. Adopting the simplified formulae from Table 11.10 and 11.8 in the HSE Guidance Notes the corresponding 50-year values can be deduced:

<table>
<thead>
<tr>
<th>Parameter</th>
<th>100-year value</th>
<th>50-year value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Storm still water - SWL (m)</td>
<td>184.83</td>
<td>182.46</td>
</tr>
<tr>
<td>Wave height (m)</td>
<td>29.25</td>
<td>27.9</td>
</tr>
<tr>
<td>Crest ratio</td>
<td>0.600</td>
<td>0.572</td>
</tr>
<tr>
<td>Deck elev. above N-year SWL (m)</td>
<td>13.55</td>
<td>15.92</td>
</tr>
<tr>
<td>Air gap (m) (= deck elev. - crest ratio x wave ht)</td>
<td>-4.0</td>
<td>-0.0</td>
</tr>
</tbody>
</table>

On that basis the ‘acceptable’ design case would have corresponded to a 1.5m higher deck elevation than the base case in Table 7.3, i.e. for a deck elevation of 15.05m above the 100-year storm still water level (17.42m above the 50-year SWL). The base case scenario therefore corresponds to some 1.5m sea-floor subsidence since design or an underestimate in environmental parameters for the design return period.

It can be seen from Table 7.3 that the ranges of wave height, crest ratio and deck elevation embrace alternative “acceptable” designs. However it should be emphasised that the purpose of this study is to provide quantitative comparisons for the available calculation methods. The reliability implications of the model selections are presented in Section 8.2. Specific conclusions regarding failure probabilities for individual installations fall beyond the scope of this initial study.

For the quantitative comparison of wave-in-deck load calculation methods, a series of analyses has been undertaken for each of the four selected models (Section 17, Shell, Statoil and Kaplan). The results of the analyses are reviewed individually in the subsections below to identify the features captured by each method. The comparison follows in Section 7.4.

### 7.3.1 API RP2A Section 17 - Example calculations

The Section 17 method determines wave-in-deck load based on a drag formulation for the maximum crest height encountering the full projected area of the deck. Calibration of the effective drag coefficients may be considered to account to some degree for the interactions. The results presented assume a wave kinematics factor of unity but a current blockage factor of 0.8. Were this set to 1.0 the forces would increase slightly (2.5%).

**Wave Height**

Figure 7.2 presents the increasing deck load with wave height for the base case structure assuming the deck to be heavily equipped or solid which is probably more appropriate to a North Sea structure than the bare or moderately equipped Gulf of Mexico categorisation. Particle kinematics are based on Stokes 5th as determined from Figure 4.1 giving a relative crest height of 0.589 (i.e. 17.24 / 29.25) compared with the 0.6 assumed for the base case in other methods. Clearly the broadside heading gives larger lateral loads. The same nonlinear relation with wave height is evident, as noted previously.
Figure 7.2
Section 17 variation in lateral load with wave height
Structural configuration
The influence of the different drag coefficients presented in Section 17 on the force level is indicated in Figure 7.3 for the base case wave height of 29.25m. It is to be recalled that the range from bare to heavily equipped decks relates to Gulf of Mexico test simulations. The results are presented with reference to the deck elevation above storm still water level; the base case corresponds to the 13.55m clearance.

![API RP 2A - Section 17 model](image)

**Figure 7.3**
Section 17 variation in lateral load with deck height and equipment density

Crest / wave height
The Section 17 methodology does not quantify variations due to the relative crest height, \( \eta \), as reliance is placed on the associated wave theory. To give some indication of the sensitivity to crest / wave height assumptions and to consider the implications of adopting linear theory, Figure 7.4 presents results of alternative calculations. The results relate to the base case (\( H = 29.25m \), deck elevation = 13.55m, \( C_d \) for heavily equipped configurations) and
the triangle is taken from the previous calculations using Stokes 5th wave theory. The alternative lines base the horizontal particle velocity on Airy wave theory in two ways:

- By considering a symmetric wave (as intended in the theory) but applying it to an asymmetric wetted area related to the crest elevation;
- By specifying a wave height corresponding to twice the assumed crest height in the Airy calculation and again applying it to an asymmetric wetted area related to the crest elevation.

It can be seen from Figure 7.4 that the force predictions are sensitive to the wave theory. However use of the Airy theory with the adjusted wave height (as in the second approach) gives a reasonable but unconservative approximation to the Stokes 5th result (-25%). Whilst this may seem significant, there is a wide spread in the forces calculated depending on crest height assumptions. Furthermore, uncertainty in the effective density of topsides structure and equipment can influence results to this degree (see Figure 7.3 and further comments related to the Shell model in Section 7.3.2).

Figure 7.4
Sensitivity to wave theory and crest height assumptions

53
Deck solidity
The final variable from Table 7.3 concerns the solidity of the deck. However this has no influence on results for the Section 17 method as only horizontal forces on the projected face area of the deck are considered.

7.3.2 Shell Method - Example Calculations
In performing wave-in-deck load calculations, use was made of the EXCEL spreadsheet provided by Shell to this study. The method is based on the momentum flux into a control volume comprising the deck and assumes all momentum is lost. On that basis no coefficients are required to account for the deck configuration; however the degree of underdeck porosity determines the flow that can pass upwards into the control volume only to be stopped by vertical faces generating a horizontal load within.

As a result, the horizontal load comprises two parts: loading on the front of the deck and loading from the passage of water from the underside. The total force therefore varies as the wave passes through the structure, as illustrated in Figure 7.5. The phasing can be seen clearly, particularly for the end on wave where the length of the structure is longer compared with the wave length than for the broadside attack direction. The deck force extracted for the subsequent comparisons is the overall maximum in both cases.

![Figure 7.5](image)

Shell method - Phasing of lateral load as the wave passes through the structure

Crest / Wave height
The increase in lateral load with wave height is clear from Figure 7.6. The Shell model (Section 6.3) is based on the use of Airy wave theory as a simplifying approximation. A kinematics enhancement factor on horizontal particle velocities is allowed for in the formulation but does not introduce asymmetry in the wetted profile, ie Equation 6.9. The sensitivity to alternative interpretations is indicated in Figure 7.7.
Figure 7.6
Shell method - Variation in deck force with wave height

For consistency between comparisons in this study, it is to be recalled that the base case is taken to correspond to a crest height comprising 60% of the wave height (H). Four potential interpretations within the Shell model are examined:

A. A symmetric wave, where amplitude is 0.5H and no kinematic enhancement factor is included.

B. As above but with a reduction in air gap corresponding to 10% of the wave height to account for the assumed asymmetry.

C. As above (i.e. \( \eta = 0.5H \), deck height reduction 0.1H) and including a wave kinematics enhancement factor of 20% on amplitude affecting horizontal velocity calculations.

D. A symmetric wave with amplitude=0.6H, thereby affecting horizontal and vertical velocities calculated but also automatically allowing for crest encroachment of air gap.
Figure 7.7 clearly shows how without any allowance for the asymmetry at the crest position (A) the degree of inundation would be underestimated (compare with B). Allowing for the velocity enhancements in the crest (A) over and above the symmetric Airy values (B) is significant. The alternative magnification of the wave as in D can be seen to deliver comparable force levels indicating that the enhancement of vertical velocities is of negligible significance. Throughout this section and in the comparisons in Section 7.4, the Shell model is used without a kinematics enhancement factor and Airy theory based on an amplitude equal to the crest height ratio (e.g. 0.6) times the wave height.
Deck solidity
Figure 7.5 illustrates the substantial contribution of loading from the underside of the deck to the overall force for the base case under consideration. Clearly the relative contribution will vary with the degree of inundation. A further influence is the proportion of wave loading that can pass up through the deck on account of the deck construction. Figure 7.8 shows the significant reduction in total lateral load if the effective porosity limits the flow down to 20% through the deck. The condition of zero porosity corresponds to deck loading only on the front face. Wave loading on the deck 'obstacle' is considered to generate increasing upward vertical loads at the same time; whilst the Shell method could be adapted to calculate these forces by the same principles, the solutions are not currently available.

![Graph showing Shell model - Influence of effective deck porosity on total lateral load](image)

**Figure 7.8**
Shell model - Influence of effective deck porosity on total lateral load

7.3.3 Kaplan Model - Example Calculations
The Kaplan model is more sophisticated in its modelling of the interaction between wave loads and deck structure and equipment and therefore provides a more detailed description of the range of forces involved.

Crest / Wave height
Figure 7.9 indicates the variation in horizontal and vertical forces with increasing wave height. The rate of increase of lateral loads with increasing wave height reduces for crest heights exceeding the height of deck beams, due to reduced velocities at the deck beam elevation. The reduction in the rate of increase does not occur when the underdeck beams are enclosed by the tank deck. The sense of the vertical loading is influenced by the phase of the wave. Both upwards and downwards forces can be generated and the maxima are extracted separately. These can be particularly important to the utilisation of the support legs already taking increasing load due to the lateral wave action.
Figure 7.9
Kaplan model - Variation of horizontal and vertical deck loads with increasing wave height
The relative height of the crest within the wave influences the calculated particle velocities and degree of inundation as shown in Figure 7.10.

Figure 7.10
Kaplan model - Influence of crest / wave height

Deck configuration
Figure 7.10 also illustrates the influence of cladding (or vertical walling) above the deck beams. In the base case it is only the under-deck beams which attract lateral load and the additional load due to the cladding above can clearly be seen.
Figure 7.11 provides further insight to the drag contribution to the loads calculated. Where a coefficient of 2.0 is considered for the base case, it can be seen that the horizontal force components reduce linearly with the assumed coefficients. In the vertical sense the drag contribution is a smaller proportion of the overall force and the results are less sensitive to the assumed coefficients.

**Figure 7.11**
Kaplan model - Variation in deck load with drag coefficients
Figure 7.12 demonstrates the significance of cladding of the deck structure above the underdeck beams. Furthermore, the figures illustrate the effect when the underside of the underdeck beams is plated creating a tank deck. The lateral loads reduce significantly as loading is only attracted on the front face. However, the vertical impact of the waves on the tank deck occurs at lower wave heights. It can be seen that when the base deck is not cladded the lateral load reaches a maximum when the inundation corresponds to the beam depth of 1.6m. It can subsequently reduce as the velocity in the mid portion of the wave impacting the beam is less. In the presence of cladding, the lateral force continues to increase, albeit at a reduced rate, on account of the reducing ‘average’ velocity. This underlines the sensitivity of force calculations for small degrees of inundation.
Figure 7.12 also illustrates the potential reduction in loads where the underside of the under-deck beams is plated to create a tank deck such that lateral loads are only generated by wave impact with the external beams. The loading associated with cladding above the deck is unchanged but proportionally has a more significant effect. Importantly it should be noted that the Section 17 projected area method would not distinguish between the tank and base deck configurations potentially underestimating the lateral loading significantly when multiple under-deck cross members are exposed. Clearly the vertical deck loads are unaffected by the degree of cladding to the walls of the deck structure.

The upward forces on the underside of the deck, $F_u$, max, increase linearly with the degree of inundation. The vertical downward force, $F_v$, min, which is essentially an inertia load, reaches a limiting value with increasing inundation. For both the upward and downward loads, the differences between the base deck and tank deck results are primarily due to buoyancy effects in the latter case.

**Deck Solidity**

With the separate treatment of lateral and vertical loads in the Kaplan model, only the latter are influenced by the degree of porosity. Figure 7.13 shows how the magnitude of vertical loads reduces radically for even a small degree of permeability reducing the slamming effects.

**Figure 7.13**
Kaplan model - Influence of deck porosity on vertical loads
7.3.4  **Statoil Method - Example Calculations**

Lateral loads calculated using the Statoil model are presented in Figure 7.14 for the range of assumed wave heights. Figure 7.15 shows the variation depending on the relative crest elevation assumed, although the base case (η=0.6) compares well with the value of 0.62 calculated using the method due to Haver[27]. Particle velocities are taken to comprise wave and current velocities. The Statoil formula for crest height is applied for illustrative purposes (Equation 6.17) although the case falls outside the specified depth range. The variation of lateral load with deck height is shown in Figure 7.16. It should be noted that the porosity of the deck is not relevant to the calculations.

**Figure 7.14**

Statoil model - Variation of lateral load with wave height
Figure 7.15
Statoil model - Variation of crest height on lateral loads
7.4 COMPARISON OF EXAMPLE CALCULATIONS

A number of observations are clear immediately from the example calculations presented above:

- Vertical load components (upwards and downwards) are only considered in the Kaplan model although the Shell model could be adapted to include loss of vertical momentum.

- The API Section 17, Kaplan and Statoil models are all dependent on calibrated coefficients. The Shell model is not, but is conservative in assuming complete momentum loss.
Quantitative comparisons for the lateral loads calculated are presented in Figure 7.17 to 7.19. The comparison assumes that the deck structure is clad or densely equipped so there is no limit on the loads calculated.

**Figure 7.17**

Lateral load comparisons with wave height
Figure 7.18
Lateral load comparisons with crest height
The immediate observation is the very significant difference in the magnitude of forces calculated. However in all cases the trends are reasonably consistent. The greatest variability in percentage terms relates to small degrees of inundation which may be expected to have significant influence on structural reliability.

In order to understand the reasons for the differences in wave-in-deck force calculated, it is important to restate some of the assumptions underlying the use of each method. For example, with regard to wave crest velocities and relative crest heights, the various model assumptions were:
- Shell - Airy wave theory with amplitude = 0.6H
- Statoil - Parametric formulae approximating Stokes' theory
- API - Stokes' wave theory

In the latter two cases, the calculated crest height ratios are in fact 0.618 and 0.589 respectively, diverging slightly from the 0.6 base case assumption adopted in both the Shell and Kaplan calculations.

In Figure 7.18 the sensitivity to crest / wave height assumptions is considered (somewhat artificially) for Statoil (using the assumed crest height in the velocity formula) and for API by adopting Airy wave theory for an amplitude ηH. The Stokes' base case values are shown for comparison on the figure. The change in wave theory may be considered to shift the results downwards.

In all cases the sequence of results lowest to highest for the base case is:

<table>
<thead>
<tr>
<th></th>
<th>0°</th>
<th>90°</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lowest</td>
<td>API</td>
<td>API</td>
</tr>
<tr>
<td></td>
<td>Kaplan</td>
<td>Statoil</td>
</tr>
<tr>
<td></td>
<td>Statoil</td>
<td>Shell</td>
</tr>
<tr>
<td>Highest</td>
<td>Shell</td>
<td>Kaplan</td>
</tr>
</tbody>
</table>

Neglecting the Kaplan method, the results are in the same sequence and Table 7.4 shows why.

Table 7.4
Comparison for base case

<table>
<thead>
<tr>
<th>Method</th>
<th>Formula</th>
<th>Base case Fx (MN)</th>
<th>Constant Coefficient</th>
<th>U (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Statoil</td>
<td>( \sqrt{\rho} C_{sw} (U + U_c)^2 A )</td>
<td>20.1</td>
<td>( C_{sw} = 3 \times \frac{1}{4} )</td>
<td>9.73</td>
</tr>
<tr>
<td>API</td>
<td>( \sqrt{\rho} C_s (U + 0.8U_c)^2 A )</td>
<td>13.0 (13.4 for ( \alpha_{st} = 1.0 ))</td>
<td>( C_s = 2.5 \times \frac{1}{4} )</td>
<td>8.62</td>
</tr>
<tr>
<td>Shell</td>
<td>( \rho(U + U_c)^2 A + \text{underdeck components} )</td>
<td>10.70 + 20.52 = 31.22</td>
<td>1</td>
<td>8.62</td>
</tr>
</tbody>
</table>

In all cases, \( \rho = \text{constant,} \quad U_c = 0.715 \text{ m/s} \quad \text{heading} = 0° \) \( = 30 \times 4 \quad = 1025 \text{ kg/m}^3 \quad = 120\text{m}^2 \)

- The Statoil and API methods have similar formulations although conceptually they differ. If current blockage were neglected, using API, the force would only increase by ~2.5%. It is a reasonable observation that the inclusion of wave kinematics and current blockage factors perhaps imply a disproportionate level of accuracy in the API model. Nevertheless, the baseline calibration has included these effects and is consistent with the assumed loading recipe. More significant is the alternative basis for wave velocity calculation but over and above that the
slamming versus drag values recommended (3 and 2.5 respectively) will always give Statoil loads some 20% higher than API.

- The Shell lateral load comprises two parts. In this case the load on the front face is less than the Statoil or API silhouette loads which is to be expected as the API drag coefficient is effectively calibrated for the 3D platform effects. The underdeck component for lateral loads as waves move up into the structure from below is significant and causes the total load to exceed the API or Statoil values. Given the underlying assumption that all momentum is lost, this upper bound is to be expected. However the basis for the Kaplan loads being even higher in the 90° case (see Figure 7.18) is not immediately clear and further explanation is provided below.

It should be noted that if there were a solid tank deck so lateral load was only generated on the front face, the Shell model would give lower load predictions than API or Statoil.

- There is consistency between the simpler models for loads in the two directions as clearly the relations are governed by the 30 : 50 dimension ratio (Figure 7.1). For the Kaplan model, base case lateral loads of 72.7 MN (90°) compare with 23.1 MN (0°); and again further explanation is provided below.

It should be noted that although the Shell model happens to fit the ratio for this geometry, this would not necessarily be the case for different inundation / aspect ratio combinations.

The Kaplan model differs from the others considered here in that the loading from each obstacle is considered. In the case study calculations no account has been taken of velocity blockage and water particle velocities at each deck beam location were calculated as for an undisturbed wave. By contrast the Statoil and API coefficients consider a macro effect of blockage in the calibrations and the Shell approach assumes that the first obstacle destroys all velocities. Neglecting velocity blockage is always conservative and can be very conservative in cases where the deck geometry presents many 'cross members'. This is the case for the broad-side waves in the case study (see Figure 7.1) where there are two perimeter beams and eight interior cross beams.

It is because velocity blockage has been neglected in the Kaplan model that it gives loads larger than the Shell model for broad-side waves. This is also the reason why the Kaplan model gives a three times larger load for broad-side as for head-on even though the frontal area ratio is 1.67. The ratio of drag areas broad-side to head-on is: 10 x 50 / 4 x 30 = 4.2.

In Kaplan's original development of the model theory, procedure and software for calculating wave-in-deck loads, velocity blockage was not addressed because it was not relevant for the geometry of the decks in the wave tank tests. Kaplan later proposed a procedure for accounting for velocity blockage which entails a 'lumping' or grouping of several cross members, with a reduction factor calculated for each group. Results depend on how members are grouped and a degree of expert judgement is required. Referencing previous in-house examples where the width and deck beam spacing are not dissimilar to the case considered here, Kaplan in private communications proposed modelling the front face plus three groups. The net result of effective velocity blockage was a 49% reduction in drag force on the (eleven) interior beams. For the eight interior beams in the present study, a 40% reduction is considered to be reasonable and the development of the resulting forces for the base case is illustrated below:
total lateral load base deck w / cladding (no blockage): 72.7 MN
total lateral load tank deck w / cladding (no blockage): 15.1
drag load interior beams (no blockage) = 72.7 - 15.1 = 57.6
drag load interior beams with blockage = .60 x 57.6 = 34.6

==> total load with blockage = 15.1 + 34.6 = 49.7 MN

which is marginally smaller than the Shell estimate.

No graphical comparison is made of the effects of deck porosity on wave-in-deck loads. Both the API Section 17 and Statoil models as presented account only for impact with the frontal area of the deck. However the Kaplan model gives a significant reduction in vertical slamming loads with porosity and accounts for downward inertia loads but neglects the lateral loads from the water passing into the deck. By contrast, although the vertical load components are neglected (but could be included) in the use of the Shell method, the lateral load increases for increasing porosity, providing a gradual transition from the plated to unplated underdeck conditions.

The foregoing comparison of the wave-in-deck load calculation methods shows that the different results derive from the alternative assumptions regarding force development. Largely excluded from the figures is the additional uncertainty in actual crest heights, with only an indication of the effects when different wave theories or crest height ratios are adopted.

The studies which follow in Section 8 examine the potential significance of these uncertainties on system reliability.
8. IMPLICATIONS OF WAVE-IN-DECK LOADS FOR STRUCTURAL RELIABILITY

In the context of the present study of wave-in-deck load calculation models, the reliability questions are firstly:

- To what extent can wave-in-deck loads affect system reliability?

and secondly

- Is the choice of wave-in-deck load model a significant influence on the calculated reliability?

This is demonstrated in Section 8.2 for the specific case study from Section 7. However other evidence from the literature is gathered in Section 8.1.

8.1 PREVIOUS STUDIES

A number of studies have now been published demonstrating the significance of wave-in-deck forces to system strength evaluations particularly in the wake of Hurricane Andrew (e.g. Reference 24). However, only limited reference has been given to the implications for reliability. Tormans and van de Graaf20, for example, report an increased annual failure probability of 2.7x10\(^{-9}\) (1 in 370 years) for the SP62-B jacket in the Gulf of Mexico when wave-in-deck loads are accounted for as opposed to 6.3x10\(^{-7}\) (1 in 1600 years) when they are not. Wave-in-deck loads were based on the ‘Shell’ model described in Section 6.3.

More detailed insight to the underlying influences is provided by Dalane and Haver77, and Haver77 using the simple Statoil model. Overall they comment that for most North Sea structures with a reserve collapse strength greater than the design level by a factor of 2-4, the most probable cause of failure would be associated with wave loading in the deck. The basis of the argument is that for existing structures the crest of a wave large enough to cause collapse would almost inevitably reach the deck, potentially changing the mode of structural failure. On this basis considerations of wave-in-deck loading are essential in system reliability calculations.

Within their specific studies a jacket in the Ekofisk area was considered first with the calculated wave-in-deck loads applied simply as four horizontal nodal forces at the top of the jacket legs (vertical effects neglected). Once the waves impact on the large beams supporting the deck, the wave-in-deck loads calculated increase (as shown in Figure 8.1) but can be seen to reach a limit as the crest goes beyond the top of the beams and the lateral load generated by particle velocities further down the wave is lower. The effect is exaggerated in the specific model because no deck structure is allowed for above the beams. In reality there would be some obstacle at the crest level, so a limit load rather than peak load is probable (e.g. Figure 7.12).

The analyses showed that the consequences of this was for the initial sensitivity of system reliability on the wave height (and hence wave-in-deck loads) to reduce and be replaced by an increased dependence on uncertainties in the load calculation method and capacity, corresponding to the limit condition in Figure 8.1.
Figure 8.2. Annual probability of failure.

Exceedance Probability

Figure 8.3. Load-displacement curve for horizontal load on deck.

Wave Load on Deck (MN)

Global Displacement (m)

Probabilistic analysis also demonstrated that the air gap coefficient is governed by the randomness of the wave and water height, whereas uncertainties in tide and surge are unimportant. Considering joint probability environmental data, the long-term uncertainty in wave height counteracts the benefit in reduced still water level as shown in Figure 8.2.

The explanation lies in the tuning dominance of the problem (i.e. load = H) and the direct dependence on load on the wetted area and hence wave height (i.e. load = H).

Figure 8.4. Flowchart for potential energy.
In terms of failure probabilities Figure 8.4 is reproduced illustrating the importance of including wave-in-deck impact at all which dominates over uncertainties $X_1$ and $X_2$ in base shear and deck load. Similarly Figure 8.5 shows the insensitivity to the slamming coefficient adopted in the Statoil model (see Section 6.4). The significance of selecting wave height and crest height is examined as illustrated in Figure 8.6. Whilst the primary importance of crest height is clear\(^{(17)}\), the practical difficulties of combining non-Gaussian processes with regular wave theories is acknowledged. For jacket evaluations wave theories are recommended but concern regarding the applicability of crest definitions for evaluating local damage is noted.
For the purposes of the present study a particularly important finding was that even for a COV of 30% in the deck load, the contribution to overall uncertainties was just 2% and failure probabilities are still governed by the inherent randomness of the annual largest wave. This suggests that greater attention should be given to the potential bias within the wave-in-deck impact loads, accepting the statistical variability.

These conclusions from the literature are combined with the specific investigations in Section 8.2.
8.2 CASE STUDY RELIABILITY CALCULATIONS

The results from the case study presented in Section 7 are applied in reliability analyses to assess to the significance of wave-in-deck loads, and the choice of load model, on the calculated system reliability.

8.2.1 Reliability Model
Two limit state functions are considered representing overall jacket failure or failure of the deck legs:

\[ g_1(z) = \gamma_s JC(h) - (\gamma_s Q_j (h) + \gamma_d Q_d (h) + Q_w) \] (8.1)

\[ g_2(z) = \gamma_s DLC(h) - (\gamma_d Q_d (h) + Q_w) \] (8.2)

The annual probability of failure derives from both modes (see also Figure 8.8) and is given by:

\[ P_f = Pr \{ g_1(z) \leq 0 \} \cup Pr \{ g_2(z) \leq 0 \} \] (8.3)

The parameters and variables are:

\[ \begin{align*}
g(z) &= \text{limit state (failure) function} \\
z &= \text{vector of random variables} = \{ h, \gamma_s, \gamma_d \} \\
h &= \text{extreme annual wave height} \\
JC(h) &= \text{jacket capacity (base shear at collapse)} \\
DLC(h) &= \text{deck leg capacity} \\
Q_j &= \text{wave and current loads on jacket} \\
Q_d &= \text{wave and current loads on deck} \\
Q_w &= \text{wind loads on deck} \\
\gamma_s &= \text{uncertainty parameter, resistance} \\
\gamma_j &= \text{uncertainty parameter, jacket wave loads} \\
\gamma_d &= \text{uncertainty parameter, deck wave loads}
\end{align*} \]

Further details are given below.

**Extreme annual wave height**

The extreme annual wave height is modelled by a Weibull distribution:

\[ F_h(h) = 1 - \exp \{- \delta (h - \epsilon)^\beta \} \] (8.4)

with parameters \( \delta = 0.086, \epsilon = 11.7 \) and \( \beta = 1.53 \). This wave height model is a close approximation to the Ekefisk model employed by Haver(19,77). This model predicts 100 year and 1000 year wave heights of 25.14 and 29.22 which, with a constant crest / wave height ratio of 0.6, give corresponding crest heights (above SWL) of 15.1 and 17.5 m, respectively.

**Jacket capacity**

The 'jacket capacity', JC, is the global base shear at platform collapse (from all loads but excluding failure at the deck / jacket interface), and may be governed, for example, by brace buckling, joint failure, foundation failure, or a combination of these. The jacket capacity is usually estimated by nonlinear pushover analyses. Often, only one pushover analysis is
performed incrementing the load pattern corresponding to the 100 year wave. However, this is potentially misleading in that the JC may be quite sensitive to load pattern, and may exhibit significant reductions with increasing wave height if alternative ‘jacket’ failure modes are triggered. The following analyses consider both cases with a constant jacket capacity and with a decreasing capacity.

Deck leg capacity
The deck leg capacity, DLC, is expressed as the global shear at the base of the deck legs at failure and may also be estimated by nonlinear pushover analyses. For the analyses presented here, constant values (i.e. independent of wave height) of DLC are assumed implying a single failure mode. However, as the deck leg capacity is typically sensitive to vertical loads, parameter studies are presented showing the effect of reduced deck leg capacity on the calculated reliability.

Wave and current loads on jacket
A power relation is assumed for the wave and current loads on the jacket:

$$Q_j = a_j h^{b_j}$$ (8.5)

with a constant exponent $b_j = 2.1$ considered to be representative for drag dominated jackets and $a_j$ representing the magnitude of the jacket load attracted.

Wave and current loads on deck
Figure 7.17 presents a comparison of the lateral load on the deck calculated by four different load models: Shell, API Section 17, Kaplan, and Statoil. It is seen that, with the exception of the Kaplan model for broad-side waves, the variation in deck load with wave height is approximately linear. Hence, wave-in-deck loads are modelled by:

$$Q_d = a_d (h - h_o)$$ (8.6)

where $h_o$ is the minimum wave height impacting the deck. The parameter $a_d$ is varied to represent the different rate of change of deck force with wave height calculated by the models. A factor of 3 is chosen which gives wider bounds than the loads predicted by the various models.

It may be noted that the nonlinear variation predicted by the Kaplan model for the 90° wave heading is due to the significant number of large exposed deck beams for this approach direction. It is believed that this is probably not a very representative geometry, and therefore only the linear load model shown above is applied in the reliability analyses.

Wind loads
The wind load on the deck, $Q_w$, is taken to be independent of wave height. This is considered to be an acceptable simplification given that wind loads typically comprise only a few percent of the total lateral load at collapse.

Uncertainty parameters
The uncertainties in loading and resistance, $\gamma_L$, $\gamma_f$, $\gamma_r$, are modelled as lognormal random variables. They are taken to be unbiased estimates (i.e. mean values of unity). The coefficient of variation (COV) of the resistance uncertainty is set to 0.15. Following Haver (19), the COV of the model uncertainty parameter for the wave loads on the jacket is
also assumed to be 0.15. The uncertainty in the deck wave loads is taken to be 0.30, see Table 6.3 (reference also API Section 17(17) and Dalian and Hover(17)).

### 8.2.2 Model Response Predictions

Table 8.1 lists the parameter values applied to study the effect of wave load model on the calculated systems reliability, with variation in the airgap (clearance above the 100 year crest) and reserve strength ratio (RSR).

#### Table 8.1 Key to Figures 8.8 to 8.15

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value (range)</th>
<th>Comment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Jacket capacity, JC</td>
<td>100 MN</td>
<td>constant for all h</td>
</tr>
<tr>
<td>Deck leg capacity, DLC</td>
<td>50 MN</td>
<td></td>
</tr>
<tr>
<td>Airgap</td>
<td>0-5m</td>
<td></td>
</tr>
<tr>
<td>Jacket load magnitude parameter, $a_j$</td>
<td>0.0516, 0.0344, 0.0258</td>
<td>RSR = 2.0, RSR = 3.0, RSR = 4.0</td>
</tr>
<tr>
<td>Deck load magnitude parameter, $a_d$</td>
<td>[0, 4, 8, 12]</td>
<td></td>
</tr>
<tr>
<td>Wind load on deck, $Q_w$</td>
<td>5 MN, 3.33 MN, 2.5 MN</td>
<td>RSR = 2.0, RSR = 3.0, RSR = 4.0</td>
</tr>
</tbody>
</table>

Consistent with industry practice(7), RSR is defined as the ratio of the jacket pushover capacity to the 100 year load effect:

$$\text{RSR} = \frac{\text{JC}}{(Q_j(h_{100}) + Q_d(h_{100}) + Q_w)} \quad (8.7)$$

For all cases considered in these analyses, the airgap is not negative so that there is no load on deck for the 100 year conditions, i.e., $Q_d(h_{100}) = 0$.

For the analyses, the base case RSR is set to 2.0. A jacket capacity of 100 MN is selected as a representative pushover capacity for a jacket with a deck size corresponding to that in the Northern North Sea platform case study in Section 7. With this assumed RSR and JC, the jacket load magnitude parameter, $a_j$, is calculated from Equations 8.4 and 8.7.

In the sensitivity studies, the RSR is varied from 2 to 4 as shown in Table 8.1. A higher RSR may be obtained by either increasing the jacket capacity or by reducing the loading (see Equation 8.7). The latter approach is selected here, but it is recognised that the choice is arbitrary and either approach would lead to the same qualitative conclusions - indeed with corresponding adjustment to deck leg capacity etc., either approach would lead to identical quantitative results.

The value for $a_d$ in Table 8.1 is found from the deck load case study, see Figure 7.17 for broad-side waves. $a_d = 4$ corresponds approximately to the API load prediction, whereas $a_d = 12$ is somewhat larger than predicted by Kaplan's model. It is noted that the wave height just hitting the deck is 22.6m and the highest wave height considered is 29.3m.
Jacket and deck loads versus wave height, based on the formulae in Section 8.2.1 and parameters in Table 8.1, are shown in for one example in Figure 8.7. Depending on the air gap, wave height and resultant loading (Q), there is the possibility for either or both the deck leg capacity and the jacket capacity to be exceeded precipitating failure, see Figure 8.8.

Figure 8.8 plots the failure functions from Section 8.2.1 in the space of the random variables wave height and load uncertainty by way of example. The intersection of the surfaces signifies the change in failure mode in different regimes. It can be seen for example that for large waves inundating the deck, the principal failure mode switches from the jacket to the deck legs. The juxtaposition of the failure modes inevitably depends on the jacket and deck leg capacities, loading uncertainties, the wave-in-deck load calculation method adopted etc. Nevertheless, together, Figures 8.7 and 8.8 illustrate the mechanisms by which failure may occur.

![Graph showing load resistance versus wave height](image_url)

**Figure 8.7**
Example jacket and deck loading functions
RSR = 2.0, airgap = 1.5m

**Note:**
- $Q_j = 0.0516 \cdot h^{2.1} \cdot a_j \cdot h^{2.1}$
- $Q_d = 8 \cdot (h - 27.64) = a_d \cdot (h - 27.64)$
- $Q_{TOTAL} = Q_j + Q_d + Q_w$
- $a_j$, $a_d$ and $Q_w$ taken for RSR = 2.0 from Table 8.1
8.2.3 Reliability Analysis Results

To assess the sensitivity of the calculated reliability to the selection of deck load calculation method, the variation of failure probability, Pf, with airgap is plotted in Figure 8.9 for four values of the deck load magnitude parameter \( a_d \). In the absence of deck loads \( (a_d = 0) \), Pf is constant for all airgaps. For large airgaps, say larger than about 3 m (for this case with \( \text{RSR} = 2 \)), the significance of wave-in-deck loads is small, and Pf is not sensitive to the magnitude of \( a_d \). Wave-in-deck loads are increasingly important for small airgaps, especially 'inadequate' airgaps of 0 to 1 m. However, even for airgap = 0, it is seen that the more significant decision is whether or not to include deck loads rather than how they are modelled: the probability of failure increases by more than a factor of 5 with deck loads included \( (a_d = 4 \text{ versus } a_d = 0) \), while a doubling of deck loads \( (a_d = 4 \text{ to } 8) \) results in only about a factor of 2 increase in the probability of failure.
The system failure probability as a function of $a_d$ is shown in Figure 8.10 for eight values of airgap. Here again, it is seen that the largest increase in Pf occurs with the inclusion of deck loads, while the significance of the different models has a relatively smaller effect.

Figure 8.11 plots $P_f$ versus $a_d$ for three values of RSR. The relative importance of including wave-in-deck loads is greatest for high RSR; for RSR = 4, Pf increases by nearly 3 orders of magnitude with the inclusion of deck loads. For this case, a further doubling of deck loads leads to about a factor 7 increase in Pf.

**Figure 8.10**  
Variation in probability of failure with deck load magnitude parameter $a_d$  
RSR = 2. Airgap = 0, 0.5, 1, 1.5, 2, 3, 4, and 5m

**Figure 8.11**  
Variation in probability of failure with deck load magnitude parameter $a_d$  
Airgap = 1.5 m, RSR = 2, 3, and 4.
One of the main reasons why the inclusion of wave-in-deck loads results in a larger relative increase in failure probability for a high RSR platform lies in the exponentially decreasing likelihood of extreme waves (see Equation 8.4; note that any of the other commonly used wave height models, eg Gumbel distribution, would show similar trends). For a platform with a high RSR, an extremely large, and thus very unlikely, wave is required to fail the platform in the absence of deck loads. Inclusion of wave-in-deck load effects reduces the design point wave height (i.e. the most probable wave height causing failure) by several metres. Such a reduction has greatest relative effect, in terms of probability of occurrence, for the largest waves - that is, for the higher RSR cases.

For the cases considered in the present study, there are two additional reasons why the effect of waves in the deck is relatively more important for the higher RSR platform. First, the deck load magnitude parameter \( a_d \) is varied by the same amount irrespective of RSR, see Table 8.1; thus the ratio of deck to jacket loads is greater for the larger RSR. Second, the deck leg capacity, DLC, has been assumed to be constant for all RSR cases. But note that even if \( a_d \) and DLC had been adjusted in correspondence with RSR, there would still be the trend that wave-in-deck has a relatively greater effect on failure probability for larger RSR cases due to the reason cited in the preceding paragraph.

The probability of failure versus airgap for various values of RSR is plotted in Figure 8.12. The figure shows that RSR alone is not a consistent measure of structural safety; for the example, a platform with RSR = 3 and a marginal airgap is at greater risk than a platform with RSR = 2 and a 1.5 m airgap.

![Figure 8.12](image)

**Figure 8.12**

Probability of failure as a function of airgap

RSR = 2, 3 and 4; \( a_d = 8 \)

The sensitivity of Pf to variation in the deck leg capacity is presented in Figure 8.13. Obviously, DLC varies for different platform configurations, eg. number of deck legs and whether or not the deck legs are battered and/or braced. In addition, the deck leg capacity may be reduced due to a large vertically downward component of the wave-in-deck forces, see Figure 7.13 which indicates a downward force of as much as 50 MN for the example deck geometry.
For the example with $RSR = 2.0$, the deck leg failure mode does not contribute significantly for $DLC$ greater than about $25MN$. This can be seen in the constant $P_f$ lines in Figure 8.13. However, for $DLC$ less than about $15MN$, deck leg failure dominates, increasing the probability of failure. The sensitivity, and relative variation, of $P_f$ to the deck leg capacity is essentially identical for the two different magnitudes of deck load; $a_d = 4$ and $8$, for the same $RSR$. However, for the higher $RSR (= 3)$ it can be seen from Figure 8.13 that the system failure is influenced by the deck loads even when the leg capacity is greater. Furthermore, the probability of failure increases even more dramatically in the higher $RSR$ case with reduced leg capacity.

![Figure 8.13](image)

*Figure 8.13*

Probability of failure as a function of normalised deck leg capacity

As noted above, it is fairly common industry practice to represent the systems capacity as a constant value. Often this is estimated by a single pushover analysis incrementing the load pattern corresponding to the 100 year wave height without accounting for higher waves and potential deck loading. However, a more detailed investigation may show that the pushover capacity reduces as waves encroach the deck. A small study is performed to illustrate possible effects of this reduction on the systems reliability.

Two forms for the jacket capacity versus wave height are considered. The first approximates a step change in the capacity as the wave height hits the deck. The second is a linear reduction of capacity over a 3:1 change in crest height. These models are shown schematically in Figure 8.14. Reductions in capacity in the range of 10 to 50% are considered.
Results are presented in Figure 8.15 (note the linear y-axis). The increase in Pf for the sloped reduction is rather minor with only a factor of 2 increase when the jacket capacity is halved. The increase in Pf for the step reduction in JC(h) is more marked, with Pf increasing by about a factor of 3 for a 30% reduction in jacket capacity. Note that this example considered the base case with RSR = 2.0; again the relative increases in Pf would be greater for larger reserve strength ratios.
8.2.4 Conclusions from the Case Study

Two questions were raised in the introduction to this Section. In response to the first question, "to what extent can wave-in-deck loads affect system reliability?", the results of the case study support the findings of previous work. Namely, that wave-in-deck loads have a large effect on the reliability of typical jackets, and should be included in the safety assessments. The significance of wave-in-deck loads is largest for platforms with marginal to moderate airgaps; results shown in Figure 8.8 indicate that, for the example considered, the airgap needs to exceed about 4 or 5 m for wave-in-deck loads not to affect the failure probability significantly. This observation is important when set in the context of industry guidelines which stipulate a minimum 1.5 m airgap.

It is also found that wave-in-deck loads have a relatively greater affect on system reliability for jackets with large reserve strength ratios (see Figure 8.9). This is because extremely large waves, which for most structures will exceed the deck height, are required to cause failure of a robust jacket with an RSR of 3 or more. However it should be noted that this observation does not imply that wave-in-deck loads are unimportant for structures with marginal RSRs - indeed for such structures the absolute importance of wave-in-deck loads is probably greater as the probabilities of failure are higher.

The corollary is that reserve strength calculations which neglect the potential for wave loading in the deck can be misleading if used as an indicator of system reliability. The discrepancy is greatest for high jacket RSRs where wave-in-deck loads can significantly increase the probability of failure. For lower RSRs where the probability of jacket failure is greater anyway the relative influence may be less; in absolute terms wave loading in the deck still makes failure more likely.

Two observations are made with respect to answering the second question "does the choice of the wave-in-deck load model have a significant influence on the calculated reliability?". First, it is found that the estimated failure probabilities are not highly sensitive to the model used to estimate lateral loads: the effect on Pf for load models differing by a factor of two is much less than the effect of neglecting rather than including wave-in-deck loads. The second observation concerns the vertical loads imposed by waves in the deck. Ultimate capacity analyses of some deck structures have shown that the lateral load carrying capacity of the deck legs may be reduced significantly by these vertical loads [in-house studies, Offshore Design]. The study here shows that the failure probability may be increased by a factor of 3 to 5 for some cases with reduced deck leg capacity. For such structures, a wave-in-deck load model capable of predicting the vertically downward oriented inertia loads should be adopted.

A final comment concerns the estimation of the resistance compared to uncertainties in the loading. It is well known that the uncertainties in the resistance have only a minor effect on the estimated reliability of jackets under extreme environmental events. However, a bias in the estimate of the pushover capacity may have an effect on the estimated reliability. An unconservative bias may be introduced by estimating pushover capacity based on the 100 year wave load pattern, which typically does not encroach the deck. However, use of the available tools for predicting wave-in-deck loads may be applied to establish more relevant extreme load patterns. Preferably, the systems capacity as a function of wave/crest height should be established.
9. CONCLUSIONS

Wave-in-Deck Load Calculation Methods
Six wave-in-deck load calculation methods have been considered in this study with detailed quantitative comparison of four. An identical deck structure was analysed for different environmental conditions and degrees of inundation enabling the trends and absolute values delivered by each method to be examined. The basis of the models is compared in Table 6.1 with a qualitative screening in Table 7.1.

The interaction between the wave crest and deck structure is complex and data are sparse. The models necessarily involve simplifying assumptions or a degree of empiricism to quantifying the net loading effects. However this study has shown forces differing by a factor of 2 or 3 depending on the method used; in some circumstances some models differed by factors as high as 8.

Specific features highlighted in the study are presented below but the more complete explanations are contained in Section 6.

- The API Section 17 method is very simple to apply and is backed by calibrations for Gulf of Mexico model tests. Only the frontal deck area is considered and the passage of the wave through the structure altering the deck / jacket loading is neglected, as are vertical loading components. In almost all cases the API Section 17 method gave the lowest lateral loads even though the highest recommended coefficients corresponding to a densely equipped ‘solid’ deck configuration were adopted. Comparisons with field data[52] have however shown the method to ‘predict’ instances of failure due to deck inundation.

- The Statoil model is similar in format to the API Section 17 approach but with higher ‘slamming’ coefficients than the Section 17 ‘drag’ values, resulting in slightly higher lateral loads.

- The Shell model assumes the deck defines a control volume and the momentum of all water particles entering that volume is lost, thereby generating load on the structure. impact on both the front face of the structure and due to the passage of water into the underside of the control volume is considered, introducing a phasing into the total forces calculated. The model is tractable in spreadsheet form and, although not examined here, is readily adaptable to include upward as well as lateral loading effects. Given the upper bound implicit in the concept of complete momentum loss, it is to be expected that the forces calculated in the case studies exceed either the Statoil or API Section 17 values. The trends are however consistent and in good agreement.

- The Kaplan model provides the most sophisticated representation of wave-in-deck load mechanisms, accounting for drag, inertia, slamming and buoyancy effects on specific obstacles depending on orientation and geometry. As a result it is the most complex of the models to apply. The Amoco and detailed Chevron models are conceptually similar but were not included in the quantitative comparisons in the present study. The Kaplan model has been calibrated against test data including free-field conditions in the North Sea. Neglecting the effects of velocity blockage where multiple items obstruct the flow has been shown to be very conservative, delivering calculated forces as much as twice the Shell model values. Nevertheless with judicious grouping of elements, representative
forces are calculated. The Kaplan method accounts both for vertical and horizontal loading components, enabling meaningful assessment against the deck leg capacity. The Kaplan software is commercially available.

Implications of Wave-in-Deck Loads for System Reliability
The deterministic comparisons of wave-in-deck loads were carried through to reliability studies for idealised installations with different degrees of jacket and deck loading contributions. On the basis of these studies, conclusions regarding the significance of wave-in-deck loads for system reliability can be made as follows:

- Historic practice of adopting a prescribed 1.5 m air gap to provide a margin against waves entering the deck irrespective of environment has delivered inconsistent safety with a variable probability of occurrence.

- Meaningful evaluations of system reliability require explicit consideration of wave-in-deck loads. Even for platforms with high jacket reserve strength ratios, the relative influence of waves in the deck on failure probabilities can be considerable. In absolute terms the implications when the jacket RSR is already low may be even more significant. An unconservative bias may be introduced by factoring the design wave loading profile, neglecting the encroachment of waves into the deck. Instead system capacity as a function of wave/crest height should be established.

- The primary consideration is to account for wave-in-deck loads; even with a factor of 2 on the calculated deck force, the variation in calculated failure probabilities is secondary.

- Wave-in-deck loads can alter the mode of failure from the `jacket’ to the deck legs which have not generally been designed to resist these loads. In such cases vertical loads, reducing the available lateral capacity, can increase failure probabilities by a factor as great as 3 or even 5. This underlines the importance of predicting downward inertial loads associated with deck inundation.

Closure
The study has underlined the importance of accounting for wave-in-deck loading effects on the capacity, mode of failure and hence probability of failure of offshore installations. The variation in forces calculated with the methods is considerable but has been shown to be less significant than the decision to account for the effects.

There is no clear cut recommendation for the future model to adopt:

- The API Section 17 and Statoil models are very simple to apply and readily answer the demand to include rather than ignore wave-in-deck loads. However in the Section 17 case, for example, the global calibration of complex interactions in a drag coefficient is a gross simplification and not necessarily representative of North Sea situations.

- The Shell model has a firm physical basis, albeit conservative, and is straightforward to implement. However it is not complete and will not give insight to detailed loading effects or, more importantly, to the increased vertical loading in the deck legs due to slamming and inertial effects.
- The Kaplan model has the potential to model most accurately the detailed loading effects but does require deeper understanding of potential shielding effects and the mechanisms of fluid-structure interactions if appropriate coefficients are to be used. However where the deck legs are vulnerable it provides the only method to account for combined vertical and horizontal loading effects.

On this basis the Shell model is recommended to establish the significance of wave-in-deck loads on individual platform reliability. However in instances where detailed study of the effects of inundation is required or where there is potential for water to enter the deck, giving rise to substantial vertical loads, the Kaplan model (and Amoco / Chevron model variants) is preferred.
10. REFERENCES


29. Telecommunication between H Bolt (BOMEL) and R Mercier (Shell), 10 April 1996.


37. Metocean report for Operator. ‘Return periods of zero airgaps at Southern North Sea platforms’.


48. Health and Safety Executive. 'Offshore Installations (Safety Case), Regulations SI 1992 No 2885.


32. Offshore Design a/s. Case specific wave-in-deck loading example.


37. Metocean report for Operator. ‘Return periods of zero airgaps at Southern North Sea platforms’.


48. Health and Safety Executive. 'Offshore Installations (Safety Case), Regulations SI 1992 No 2885.


APPENDIX

HSE FIXED STEEL PLATFORM DATABASE - AIRGAP TRENDS

An additional area of study requested by the Research and Project Officers as part of this contract was for the HSE fixed steel database to be examined in relation to airgap. Where traditional guidance has stipulated airgap dimensions uniformly across all UK sectors, recent understanding of environmental variations with time have demonstrated that the probability of inundation will be higher in the Northern North Sea for the same clearance. Furthermore the 1.5m airgap requirement may not be sufficient to deliver the required platform reliability. To get a measure of the potential significance of these developments for UK installations the HSE platform database was identified as a potential source of pertinent information. The airgap for all installations was calculated, compared with Guidance provisions (1.5m) and any variations between different areas were identified. The aim of the investigation was to examine trends in airgap rather than concentrate on absolute values. Extracts from the database were provided by the project Officer and key field names, and their definition within the manual, dated 13 February 1992, are as follows:

<table>
<thead>
<tr>
<th>ID</th>
<th>Identity number [the full title was not provided by way of a confidentiality restriction].</th>
</tr>
</thead>
<tbody>
<tr>
<td>AREANAM</td>
<td>Location by area (e.g. SNS, NNS etc).</td>
</tr>
<tr>
<td>UNITS</td>
<td>Units (metric - m, kN; imperial - ft, kips).</td>
</tr>
<tr>
<td>INSDATE</td>
<td>Date of installation.</td>
</tr>
<tr>
<td>CDECZ</td>
<td>Height of cellar deck, stated to correspond to the ‘lowest level of deck structure’.</td>
</tr>
<tr>
<td>WD</td>
<td>Water depth (LAT) - ‘specified LAT’.</td>
</tr>
<tr>
<td>WHT</td>
<td>Wave height (max) - ‘Max storm height from any direction’.</td>
</tr>
<tr>
<td>TRNG</td>
<td>Tidal range.</td>
</tr>
<tr>
<td>SURG</td>
<td>Surge.</td>
</tr>
</tbody>
</table>

Whilst the above fields were sufficient to calculate airgap additional data were provided in terms of wave period, wave velocity, number of jacket legs, jacket dimensions, deck area and deck type.

To calculate the airgap simply, it was necessary to assume the relative crest height and a value of 0.6 x maximum wave height was adopted. With wave heights in some regions as great as 30m and airgaps of just a few metres the significance of this assumption must be recognised. Calculation of crest heights using appropriate wave theories or specific formulations (Jahns and Wheeler, Haring and Heideman etc) would be preferred but in light of other reservations with the database the additional effort was not considered appropriate at this stage. The assumptions may nevertheless be considered to give reasonable insight to airgap trends in UK waters. The calculation of airgap was based on:
airgap = lowest deck height - (water depth including tides and storm surge plus crest height)

= CDECZ - (WD + TRNG + SURG + 0.6 WHT)

However before examining the trends a number of base checks were undertaken on the data. From reviewing the data it appeared that there were a number of omissions and several incomplete entries. Where these were needed to calculate airgap the entry had to be rejected. The remainder of the entries were subdivided by area but otherwise listed in identity number sequence.

The 'units' column provided some confusion in that, in general, it would appear that values are given in metric units. This was verified with a few plots.

Figure A.1 compares wave height with water depth and the various regions are distinguished. Except for two SNS installations, the relation is consistent and the values reasonable. The two outliers may reasonably be assumed to be in imperial units within the database and multiplication by 0.3048 (i.e. 12x25.4/1000) brings them within the body of SNS data.

It is noted that the airgap information given in Figures A.2, A.3 and A.4 is limited to an examination of trends and should not be used to investigate absolute values.

Figure A.2 brackets the airgap calculated for the platforms and a number of observations can be made:

- The magnitude and distribution of airgaps across the regions is similar.
- A number of very large airgaps (>12m) are calculated both in SNS and NNS regions. Whilst there may be operational reasons the potential for errors within the data should be remembered.
- Two platforms, both installed in the mid 1970s, have airgaps less than zero. Data errors are suspected but further investigation would be appropriate.

An alternative presentation is given in Figure A.3 where, excepting the anomalies already noted, a consistent spread of airgap across the water depths can be seen. In Figure A.4 an attempt is made to identify any changes over time with respect to airgap provision. Slightly higher airgaps can be seen on average for 1990s installation but the trend is not marked. Given the omission of installation year for some database entries, this figure represents a further subset of the platforms.

The design values within the database are quoted for different return periods. The return period is generally 50 years although 100 year values are cited for a few platforms with installation dates from the 1960s to the present day. Miscellaneous values cover one instance for which a 30 year return period is quoted and another case where no value is given. Separating out the 50 year and 100 year design data does not reveal any difference in average airgap values, suggesting that no specific evaluation of criteria with respect to return period has been undertaken.

An important stage in reviewing the database would be to develop quantified statistics in relation to airgap. However, without an initial validation of the data the exercise would imply greater accuracy than can be attributed at this stage. Nevertheless it is recommended that the work be pursued.
Figure A.1
HSE database - Initial comparison of wave height with water depth by region

Figure A.2
Comparison of airgap by region
Figure A.3
Comparison of airgap with water depth by region

Figure A.4
Comparison of airgap with year of installation by region