A REVIEW OF SYSTEM RELIABILITY CONSIDERATIONS FOR OFFSHORE STRUCTURAL ASSESSMENTS

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TABLE OF CONTENTS

1. Project Summary ........................................................................................................ 4

2. Introduction .................................................................................................................. 5

3. Generic reliability issues ............................................................................................. 6
   3.1 Uncertainty ............................................................................................................... 6
   3.2 Sensitivity ................................................................................................................ 7
   3.3 Distribution ............................................................................................................. 7
   3.4 Structural modelling ............................................................................................. 7
   3.5 Confidence ............................................................................................................ 9
   3.6 Improving consistency ......................................................................................... 10
   3.7 Targets for reliability ......................................................................................... 10

4. Uncertainty and sensitivity .......................................................................................... 14
   4.1 Qualitative descriptions of uncertainty ................................................................. 14
   4.1.1 Non-cognitive uncertainty ............................................................................... 14
   4.1.2 Cognitive uncertainty .................................................................................... 15
   4.2 Mathematical description of uncertainty ............................................................ 16
   4.3 Modelling uncertainty ....................................................................................... 18
   4.3.1 Sensitivity studies ......................................................................................... 20
   4.3.2 Generic uncertainty and sensitivity considerations ....................................... 23

5. Structural Models ......................................................................................................... 25
   5.1 Advancement of computer models .................................................................... 25
   5.2 Specific examples of models and analyses undertaken using different software ..... 26
     5.2.1 Analyses undertaken using ABAQUS ......................................................... 26
     5.2.2 Analyses undertaken using USFOS ........................................................... 28
     5.2.3 Analyses undertaken using RASOS ........................................................... 29
     5.2.4 Analyses undertaken using SAFJAC ......................................................... 30
   5.3 Distribution .......................................................................................................... 30
   5.4 Structural modelling ........................................................................................... 31
   5.5 Confidence .......................................................................................................... 31
   5.6 Improving consistency ....................................................................................... 31
   5.7 Targets for reliability ......................................................................................... 31

6. Environmental parameters .......................................................................................... 33
   6.1 Waves .................................................................................................................... 34
     6.1.1 Wave theories .............................................................................................. 34
     6.1.2 Wave forces ............................................................................................... 36
   6.2 Current ................................................................................................................. 38
   6.3 Wind ..................................................................................................................... 39
   6.4 Extreme environmental event methodologies ..................................................... 40
     6.4.1 NOCDAP, 1985 .......................................................................................... 42
     6.4.2 Metocean Plc, 1990 .................................................................................... 42
     6.4.3 Wen and Banon, 1991 ................................................................................. 43
     6.4.4 Shell, 1994-6 .............................................................................................. 43
     6.4.5 Paras Ltd, 1996 .......................................................................................... 45
     6.4.6 Concluding remarks .................................................................................... 46
   6.5 Environmental uncertainties and sensitivities ..................................................... 46
     6.5.1 Uncertainties ............................................................................................... 46
     6.5.2 Sensitivities ............................................................................................... 48
   6.6 The ‘airgap’ issue ............................................................................................... 49

7. Foundation Modelling ................................................................................................. 50
   7.1 Probabilistic foundation modelling methods ....................................................... 51
     7.1.1 Gilbert and Tang Approach......................................................................... 52
     7.1.2 The “Confidence” Approach ...................................................................... 53
     7.1.3 Fugro’s probability approach ................................................................... 54
   7.2 Deterministic foundation modelling methods .................................................... 56
     7.2.1 Imperial College (IC) Method .................................................................. 57
8. Ultimate Capacity Predictions................................................................. 63
8.1 Different approaches to ultimate capacity prediction......................... 63
  8.1.1 Shell .......................................................... 63
  8.1.2 DNV/SINTEF ................................................. 64
  8.1.3 PMB Engineering ............................................... 65
8.2 Benchmarking studies ........................................................................ 65
  8.2.1 Hurricane Andrew JIP .......................................... 65
  8.2.2 API Task Group JIP ........................................... 67
  8.2.3 HSE / MaTSU .................................................. 68
  8.2.4 Shell ................................................................ 69
  8.2.5 Amoco ................................................................ 69
  8.2.6 BOMEL .......................................................... 69
8.3 Generic conclusions regarding ultimate capacity prediction ............... 69

9. System Effects ....................................................................................... 71
  9.1 Deterministic system effects ............................................................. 71
    9.1.1 Reserve strength .................................................. 72
    9.1.2 Residual strength ............................................... 74
    9.1.3 Redundancy ....................................................... 74
  9.2 Probabilistic system effects ............................................................... 75
    9.2.1 Reserve strength .................................................. 75
    9.2.2 Redundancy ....................................................... 76
  9.3 Generic system effect considerations ............................................... 77

10. Case studies .......................................................................................... 78
  10.1 Indefatigable 49/18AD ................................................................. 78
    10.1.1 Analyses and methodology ....................................... 78
    10.1.2 Results and Conclusions ........................................... 80
  10.2 Lomond ......................................................................................... 80
    10.2.1 Analyses and methodology ....................................... 80
    10.2.2 Results and Conclusions ........................................... 81
  10.3 Leman 49/27 AP ......................................................................... 83
    10.3.1 Analyses and methodology ....................................... 83
    10.3.2 Results and Conclusions ........................................... 84
  10.4 Hurricane Andrew JIP ................................................................. 85
    10.4.1 Analyses and methodology ....................................... 85
    10.4.2 Results and Conclusions ........................................... 87
  10.5 Summary of Case Studies .............................................................. 87

11. Offshore Reliability Approaches .......................................................... 89
  11.1 Methods ....................................................................................... 89
  11.2 Search algorithms based on probability criteria ............................. 90
  11.3 Pushover analysis assisted by simulation, sequence or response surface methods .... 91
  11.4 Simplified system reliability methods ......................................... 93
  11.5 Discussion ................................................................................... 95

12. Discussion on review study .............................................................. 97
  12.1 Summary of generic issues relating to offshore structures ............... 97
  12.2 Summary of issues specific to fixed platform type offshore platforms .. 99
  12.3 Identification of technical and philosophical issues .......................... 101

13. References .......................................................................................... 103
1. Project Summary

The main objective of this project is to develop a generic framework which will set the basis for achieving more consistent system reliability assessments. The main steps involved in a system reliability assessment, together with the key technical and philosophical issues, will be identified and examined. Their inter-relations and relative significance will be assessed in order to link them together in a rational process that will provide the basis for consistent reliability assessments. The key underlying question throughout this project is what changes/improvements can be made to reliability assessments in order to move towards more consistent reliability. The perceived benefits to the customer of this project include: providing basis for future working practice/guidance as move towards more consistent reliability, with improved preparation, improved consistency in results; along with allowing the framework to be used as application, management, quality assurance and educational/training tools.

Phase 1: Review study

The first report, “A review of system reliability considerations for offshore structural assessments”, presents the findings of a review study which aimed to identify and assess the state of the art in the area of offshore structural system reliability, as well as generic aspects of structural reliability. The overall emphasis in this review study was to identify the sensitivities and difficulties associated with reliability analysis that prevents consistent reliability predictions from being obtained.

Phase 1: Development of generic assessment & presentation framework

The second report “Development of a structural system reliability framework for offshore platforms” presents the framework for structural system reliability assessments of offshore platforms. The background to the need for the framework is briefly discussed, along with the main issues arising from the review study. A number of different formats are used for the presentation of the framework.

Phase 2: Offshore study

This third report, “A parametric and sensitivity offshore study”. Based on the findings of the review study and experience gained from the framework development phase, a number of sensitivity studies were identified. These included: yield strength and foundation capacity parametric studies, foundation capacity assessment sensitivity studies, and comparison studies of the main methods of reliability analysis used in the offshore industry.

Phase 3: Development of offshore framework

The fourth report, “Presentation of a structural system reliability framework for fixed offshore platforms” is an executive summary report. This summarises the key findings from Phases 1 and 2, and presents the revised framework in context in sufficient detail to enable the reader to apply the flowcharts and tables presented therein.
2. Introduction

This report presents the findings of the review study that was performed as the first part of the research into the development of a structural system reliability analysis framework for fixed offshore platforms. The review study aimed to identify and assess the current thinking in the area of offshore structural system reliability against environmental overload, as well as generic aspects of structural reliability. The overall emphasis in this review study has been to identify the sensitivities and difficulties associated with reliability analysis, which prevent consistent reliability (or failure probability that could begin to be interpreted as absolute values for decision making) predictions from being obtained.

Historically, offshore platforms were designed to meet a standard that had a single design level of risk and reliability. However, recent changes to the offshore safety regime have meant that it is possible that in the future, it will be common for an operator to choose a risk level for a facility at the start of design. This risk level will then be used to determine the load factors, and overall reliability of the structure relative to the forces that it must withstand. In the past, it was thought that the use of a single design level imparted a consistent level of reliability, however, this has not been found to be the case. Variability has been introduced due to the wide variation in load magnitude and an even wider variation in structural resistance due to structural form and framing [Wisch, 1997; Wisch, 1998]. Recent advances in reliability fundamentals have provided immense insight into the structural system performance. However, more investigation is required to identify and study the issues that prevent consistent reliability predictions from being made.

This report incorporates an introduction to the problem, and to generic reliability issues, and in particular, the reasons behind uncertainty and sensitivity. The subsequent sub-sections then briefly introduce all the major aspects within a reliability analysis. A number of case studies are then reported, along with an investigation into the different reliability approaches currently used offshore. A discussion and conclusions are then presented.
3. Generic reliability issues

Assessing the reliability performance of systems involves dealing with events whose occurrence or non-occurrence at any time cannot be predicted. It is not possible to tell exactly when a working component or system will fail i.e. if a group of identical components, are manufactured in the same batch, installed by the same engineer and used under identical operating and environmental conditions, all components would fail at different times. Thus, handling events whose occurrence is non-deterministic is a problem commonly experienced in many branches of engineering, and in particular, in the field of offshore engineering [Birkinshaw et al., 1994; Vugts and Edwards, 1992]. Its solution requires some means by which the likelihood of an event can be expressed in a quantitative manner - this, therefore, enables comparisons to be made as to which of several possibilities is the most likely to happen. The probability of occurrence of a specific event is a “scientific measure” of chance, which quantitatively expresses the likelihood of an event occurring.

3.1 Uncertainty

“Structural reliability is concerned with the rational treatment of uncertainties in structural engineering design and the associated problems of rational decision making.” [Thoft-Christensen and Murotsu, 1986]. Uncertainty is generally categorised into three groups: physical uncertainty, statistical uncertainty, and modelling uncertainty [Thoft-Christensen and Murotsu, 1986; Peters et al., 1993; Det Norske Veritas, 1996b]. Physical uncertainty arises from the actual variability of physical quantities, such as loads, material properties and geometric dimensions. It can only be quantified by the examination of representative sample data. The statistical uncertainty arises due to a lack of information. For a given set of data, the distribution parameters may be considered random variables, where the uncertainty of which is dependent upon the amount of sample data. The model uncertainty occurs as a result of simplifying assumptions, unknown boundary conditions and as a result of the unknown effects of other variables and their interactions that are not included in the structural analysis model [Liu et al., 1996].

A degree of uncertainty is also introduced due to the user, and will therefore be affected by the user’s level of competence in carrying out pushover type analyses and reliability analyses. The competence of the user becomes more critical to the results when the stage
being undertaken has either high uncertainty in methodology or is highly sensitive to the reliability analysis. However, this type of uncertainty is not usually included directly in a reliability analysis. An interesting study was carried out on reliability based evaluations of human and organisational errors in the reassessment and re-qualification of platforms [Bea and Moore, 1994]. However, it is outside the scope of this current research to investigate this issue further.

3.2 Sensitivity

Along with uncertainty, another fundamental factor in any structural reliability analysis is sensitivity. This involves a study of the effect each of the different parameters has on results of reliability analysis of the overall structure. A study of the sensitivities of given variables can assess their relative contributions to the overall uncertainty of reliability. If the overall effects of changing a variable are found to be small, then the variable can be treated deterministically. However, where changes in a variable are found to affect the overall reliability significantly, then it is important to model the variable by using the best available distribution.

3.3 Distribution

A distribution can be defined as the set of possible values of a random variable. There are a number of different distributions that can be used to describe certain parameters which are taken into account in the calculation of the probability of failure, including:

- Binomial
- Poisson
- Normal (or Gaussian)
- Log-normal
- Weibull
- Rayleigh
- Exponential

One of the most commonly applied distributions is the lognormal, which can be defined as the distribution of a random variable, X, when log X is a random variable with a normal distribution.

3.4 Structural modelling

Some initial studies on idealised behaviour of structures concluded that “in modelling offshore jacket frameworks a general procedure was needed that could treat realistic structure geometry (combinations of series and parallel networks), a range of behaviours (some brittle, some ductile and some in between depending on [failure] mode reached) and finally, working with basic variables to assess correlation effects” [Moses and Liu, 1992].
These basic principles can be applied to other types of offshore structures as well – including ships, jack-ups, semi-submersibles, and tension leg platforms which can all be treated in this manner.

A significant number of detailed structural models have been developed and analysed over the past few years. Sometimes a generic simplified “stick model” is used in preliminary analyses. This involves reducing the structures of a certain type into an equivalent collection of cylindrical piles and then analytically calculating the base shear or overturning moment when the piles are exposed to a simplified wave [Shell Research, 1993]. In some instances where the jacket is the key feature of the analysis, a simplified version of the topside is modelled with a coarser mesh. Where the foundations are not the key feature, they are either ignored completely or the pile supports for the jacket are modelled as a number of springs; the stiffness of which were taken from those used in an earlier fatigue analysis. Conductors are only sometimes included in the analysis, and the overall effect of their inclusion has been found to amount to approximately 2% of the reserve strength load factor, in one particular study [Brown and Root, 1995].

For the structural design of offshore platforms, both specific non-linear programs for analysis of structural collapse and more general, conventional finite element packages are used. A full description, comparison and appraisal of the available FE software packages were undertaken in 1993 and 1995 by Billington-Osborne Moss Engineering Limited (BOMEL), as part of a review of the ultimate strength of tubular framed structures [Bolt et al., 1995]. Table 1 is extracted from this report.

<table>
<thead>
<tr>
<th>Program name</th>
<th>Full Title</th>
<th>Development organisation</th>
</tr>
</thead>
<tbody>
<tr>
<td>EDP</td>
<td>Extended Design Program</td>
<td>Digital Structures USA</td>
</tr>
<tr>
<td>FACTS</td>
<td>Finite Element Analysis and for Complex Three Dimensional Systems</td>
<td>Structural Software Devt. USA</td>
</tr>
<tr>
<td>INTRA (KARMA)</td>
<td>Development of INelastic Tower Response Analysis</td>
<td>ISEC UK</td>
</tr>
<tr>
<td>SAFJAC</td>
<td>Strength Analysis of Frames and JACkets</td>
<td>BOMEL UK</td>
</tr>
<tr>
<td>SEASTAR</td>
<td>Proprietary non-linear dynamic analysis program development of INTRA</td>
<td>PMB Engineering USA</td>
</tr>
<tr>
<td>USFOS</td>
<td>Progressive collapse analysis of steel offshore structures</td>
<td>SINTEF Norway</td>
</tr>
</tbody>
</table>

Table 1: Non-linear software for pushover analysis of offshore structures [Bolt et al., 1995]
Different software packages operate in different ways, with different numbers of elements required to represent the members of a structure. For example, the basic principle behind SAFJAC and USFOS is to represent each individual member in the structure by one finite element. This is therefore able to take into account large displacements of the element. Closed form solutions are obtained for the elastic total and incremental stiffness matrices which contain all information required to identify overall buckling of members or sub-systems [Sigurdsson et al., 1994; BOMEL, 1998].

Linear or non-linear material behaviour is also an important issue. Non-linear behaviour can be modelled by means of plastic hinge theory in which the yield criterion is expressed in terms of two plastic interaction functions; one represents first fibre yield and the other represents full plastification of the cross-section. Leg and bracing members, which carry high loads in both axial and transverse directions, can be modelled as non-linear beam columns. In a WSAtkins study [Gierlinski et al., 1993], these beam columns were allowed to develop plastic hinges by yielding in tension. It is important to note that programs such as ABAQUS do not work on this basis, but use a method of distributed plasticity [Hibbitt et al., 1995]. Both intact and damaged structures can be assessed, where the pseudo damaged state can be introduced, for example by including a severe imperfection into one of the braces e.g. 0.01 of a member length [Gierlinski et al., 1993].

3.5 Confidence

For the reliability of a structure to be determined, an attempt is made to most accurately predict, for a specific structure with actual foundation characteristics in a defined location, using specific environmental condition parameters, whilst using a particular software package. In the past, reliability results were taken as an indication of the notional reliability of a structure [Frieze, 1989]. Over the past few years, however, there has been a concerted effort to bring the reliability prediction as close to “true” reliability as possible. Changes in the modelling of structures and in the software used have helped to minimise the ‘error’ incurred during the initial stages of the reliability assessment process. Progress in predicting environmental conditions has enabled more accurate representation of the environmental loads. Despite the fact that particular oil companies have claimed that ‘the true reliability’ has been achieved by the application of their specialised techniques [Vugts and Edwards, 1994], this is a view that is not generally sanctioned within the offshore industry. The majority of researchers and organisations involved in the derivation of structural reliability
are still working on the premise that the reliability derived is an indication of likely events which is useful for decision making.

3.6 Improving consistency

A comparison of the reliability of two or more structures must be approached with caution, since the data, methods and assumptions used to assess structures have changed over the recent past, and still vary from one user to another. Any comparisons undertaken must be strictly on a like-for-like basis. This aspect was studied in detail by Onoufriou in 1996, who concluded that “…there are a number of technical aspects that need to be examined more closely such as foundation modelling, joint failure, air gap as well as load application and failure criteria before we are able to make consistent and accurate pushover comparisons” [Onoufriou, 1996a].

3.7 Targets for reliability

The setting of target safety levels for the assessment of offshore structures was the subject of an important critique paper presented in 1996 [Birkinshaw and Smith, 1996]. The historical framework for assessment of offshore installations on the UKCS (United Kingdom continental shelf) was presented, along with current changes in both legislation and advances in technologies. The movement from the historical to the new regime required careful consideration of the past so that large step changes in practice were either eliminated, if found to be unsafe, or were documented such that the changes were fully understood [McIntosh and Birkinshaw, 1992; Birkinshaw and Smith, 1996]. The setting of targets for safety was also presented. It was noted that “a totally quantitative target assumes almost perfect technical knowledge and also assumes that one understands the socio-political factors that also play a part in target setting”.

The traditional method of safety factors used on the UKCS had been implicit within the legislation. A minimum safety level was incorporated within the probability of failure associated with extreme weather, set at a minimum of a 50-year condition:

\[ P(f) = P[R < S] \]

Where \( P(f) \) = probability of failure,

\( R \) = resistance as specified in the code/standard,

\( S \) = loading associated with the combination of the 50 year wind, 50 year wave and 50 year current.
The basis of the goal setting regime has allowed new approaches to the assessment of structures to be developed. New approaches are of two forms (or a mix of two forms):

- firstly, where explicit notional safety levels are calculable, and
- secondly, where calibrated codes and standards rely upon target safety levels.

Use of the first form “requires high technical competence and is not without its technical challenges, but offers many advantages to those willing to invest the effort. A better physical understanding of the installation is gained which provides benefits in the life cycles process and allows funds to be allocated to places where the best safety benefits can be achieved. It also allows for an explicit ALARP (As Low As Reasonably Practical) demonstration. Due to the uniqueness of this method, guidelines are not beneficial as they interrupt the true goal setting nature of the method. Deriving the target and performance standards to meet that target are the challenges” [Birkinshaw and Smith, 1996].

The second form is the more traditional approach. It has the benefit of “repeatability and familiarity as long as the standards do not radically alter.” Experience can be used as an aid in the calibration process of this method [Kam et al., 1993; Birkinshaw and Smith, 1996; and De, 1995].

The offshore industry is not the only industry to be developing reliability methods as an aid to assessing safety. The aviation industry is another important area where safety and risk are paramount in decision-making scenarios. Significant effort has been channelled in such areas over the last forty years: new safety standards have been set, and safety philosophies employing probabilistic safety assessment techniques have been developed and widely adopted within the aircraft industry [Sayce and Doherty, 1997].

The UK Civil Aviation Authority (CAA) has “developed new monitoring tools to provide the regulator, as well as the industry and the public, with more safety related information” [Sayce and Doherty, 1997]. An Accident Analysis Group was set up consisting of a group of experts, to systematically review world-wide fatal accidents in order to quantify the world aviation risk. The work of this group was stored in a database in order to provide a hazard information resource for use by safety specialists. The database is intended to provide source information enabling a wide variety of safety issues to be analysed such as occupant survival rates, accident causal chains, regional variations etc.; and provides important high level information on the risks to the world-wide air transport system.
Many specific safety studies have also been undertaken. For example, one such study presented a Bayesian approach to decision support for aviation safety diagnostics. This application was used for modelling uncertainty in aircraft safety diagnostics, including aircraft navigation and hydraulic sub-systems. For further information see reference [Luxhoj, 1997].

Along with the aviation industry, the motor industry is another area where risk and safety are closely monitored and studied on both system and component levels. Component reliability and the corresponding life duration is an area of increasing importance for vehicle manufacturers. For example, semi-parametric models and Bayesian analysis have been developed to enable prediction models of life duration to be assessed, in order to validate car component reliability [Raoult et al., 1997]. One significant advantage of this Bayesian statistical treatment is that it allows available information from previous similar products or experts’ opinions to be taken into account.

A significant study conducted for the aviation industry in 1997 concluded that “the ability to make rational and consistent decisions as to the tolerability of risk has perhaps not kept pace with advances in the science of prediction” [Nicholls, 1997]. The work identified the “issues to be considered in setting risk criteria, drawing on experience in aviation and other potentially hazardous activities.” The relationships between such issues were summarised and presented as a “generic decision framework” which was intended to “assist in treating a variety of cases in a rational and consistent manner” (see Figure 1).
Figure 1: “Generic decision framework” [Nicholls, 1997]

The generic risk management principles included in the framework were requirements originating from the nuclear industry: justification, limitation and minimisation, along with the ALARP principle [see also McIntosh and Birkinshaw, 1992].
4. Uncertainty and sensitivity

4.1 Qualitative descriptions of uncertainty

Uncertainty modelling and analysis in structural engineering started with the employment of safety factors using deterministic analysis, which was then followed by probabilistic analysis with reliability-based factors. However, during this transition from safety factors to reliability-based factors, structural engineers recognised that the nature of uncertainty extended beyond that which the theory of probability could strictly offer [Tveit, 1995]. Consequently, uncertainty in structural engineering was classified into objective and subjective types. The objective types included the physical, statistical and modelling sources of uncertainty. The subjective types were based on lack of knowledge and expert-based assessment of structural parameters [Det Norske Veritas, 1996b; Thoft-Christensen and Murotsu, 1986]. This classification was still deficient in terms of covering the complete nature of uncertainty. The difficulty in completely modelling and analysing uncertainty stems from its complex nature, in varying degrees which are incompletely comprehended. Engineers are used to dealing with information for the purpose of system analysis and design. Information in this case is classified, sorted, analysed and used to predict system parameters and performances. However, it is more difficult to classify, sort and analyse the uncertainty in this information, and use it to predict unknown system parameters and performances. As a first step, the true nature of uncertainty in structural engineering needs to be understood. Then, uncertainty can be classified into types and different analytical tools can be used for its modelling and analysis.

Uncertainty in engineering systems can be mainly attributed to ambiguity and vagueness in defining the architecture, parameters and governing prediction models for the systems. The ambiguity component is generally due to ‘non-cognitive’ or objective sources. The vagueness related to uncertainty is mainly due to ‘cognitive’ or subjective sources.

4.1.1 Non-cognitive uncertainty

Non-cognitive uncertainty can be broadly categorised into the following groups:

- Physical uncertainty: arises from the actual variability of physical quantities, such as loads, material properties and geometric dimensions. It can be quantified only by the examination of representative sample data. However, since sample sizes are
usually limited, some uncertainty must remain. Physical uncertainty can also be referred to as natural or inherent uncertainty, “Type I” or aleatoric uncertainty.

- **Statistical uncertainty**: arises solely because of lack of information. For a given set of data, the distribution parameters may be considered to be random variables, the uncertainty of which is dependent upon the amount of sample data. Statistical uncertainties are “Type II” or epistemic uncertainties.

- **Model uncertainty**: occurs from simplifying assumptions, unknown boundary conditions and because of the unknown effects of other variables and their interactions that are not included in the model. In the last few years, advances in the models and in structural analysis software have meant that modelling uncertainty has been dramatically reduced. Modelling uncertainties are “Type II” or epistemic uncertainties.

- **Measurement uncertainty**: imperfect instruments and sample disturbance when observing a quantity cause measurement uncertainty. Measurement uncertainties are “Type II” or epistemic uncertainties.

Work undertaken to produce a guideline for offshore structural reliability [Det Norske Veritas, 1996b] concluded that “the uncertainties in the structural behaviour are due to the uncertainties in both the structural and soil-structure stiffness properties, the damping properties and the model uncertainties coming from the mathematical idealisation of the structure. The latter model uncertainty is believed to be rather small and is included in the uncertainty model in connection with the computation of local stresses.”

### 4.1.2 Cognitive uncertainty

The ambiguity component is generally due to cognitive uncertainty. This can be broadly attributed to the following:

- **Definition of certain parameters**: e.g. structural performance (in terms of failure or survival), quality, deterioration and condition of existing structures.

- **Influence of human factors**

- **Definition of the inter-relationships between the parameters of the problem**: especially for complex systems.

A degree of uncertainty is also introduced due to the user and will therefore be affected by the user’s level of competence in carrying out pushover type analyses and reliability
analyses. The competence of the user becomes more critical to the results when the stage being undertaken has either high uncertainty in methodology or is highly sensitive to the reliability analysis.

4.2 Mathematical description of uncertainty

When there is uncertainty, the conventional approach is to make conservative estimates of the design parameters. In probabilistic analysis, the uncertainty about a variable or random variable is described by a probability distribution function. Opinions based on experience and judgement can be incorporated as subjective probability. To evaluate reliability, the event of interest (e.g. resistance > load) should be defined. Moreover, a specific probability distribution function is required for each random variable in the event of interest.

Since the distribution functions are often difficult to obtain, a method such as the first order, second moment method (FOSM) can provide a practical solution to the reliability problem. In FOSM reliability analysis, the mean or arithmetic average is intended to be a best estimate without conservatism, while the variance or standard deviation is used to represent the uncertainty. In the event that uncertainties in two or more parameters are not independent, the correlation coefficient or covariance is used to express the degree of dependence [Thoft-Christensen and Murotsu, 1986]. To translate mean, standard deviation and correlation of input variables to the corresponding mean, standard deviation and correlation of calculated results, a simple linear approximation is used.

In geotechnical engineering, for example, it is frequently necessary to revise estimates of conditions and performance based on new data. Revision of a probability estimate based on new knowledge can be modelled by Bayes’ theorem [Bea, 1996]:

\[ f'([\theta | \text{data}] \propto P(\text{data} | \theta) f(\theta) \]

Where: \( f(\theta) \) = probability distribution of a parameter, taking a value of \( \theta \) before new data,

\[ f'([\theta | \text{data}] = \text{probability distribution of } \theta \text{ afterwards}, \]

\( P(\text{data} | \theta) \) = conditional probability of having observed the new data, were \( \theta \) true.

Bayes’ theory has also been used extensively in other industries, including the aviation industry. For example, if aircraft safety inspectors observe a problem, they must begin to identify the cause for the problem quickly. The probabilities of the possible causes are important references to prioritise the search and identify the causes precisely and efficiently. In a recent study [Luxhoj, 1997], a procedure was developed where such probabilities could
be provided by a Bayesian ‘network’ which worked on the principle that when any problem is detected, the probabilities of possible causes can be determined after the safety observations are entered into the ‘network’.

The original Bayes’ linear model has been further developed in recent years [including Scheiwiller, 1997; Bigun, 1997; Aven, 1997]. In one such development, the Bayes’ model has been modified to allow weighting of different types of information according to the confidence that the structural engineer has in them, in order that the developed model has capabilities beyond the ‘classical’ inference process [Scheiwiller, 1997]. Information is divided into three main types: objective information containing measurements about properties; objective information based on results of data analysis using models which describe reality close enough and which are generally accepted; and subjective information containing the estimate of experts about properties. For each type of information, a distribution and its corresponding parameters are obtained, before the engineer has to combine the different results in order to obtain one set of parameters.

The developed Bayes’ model contains five main steps [Scheiwiller, 1997]:

- **Choice of stochastic model** - The distribution type and the sample are assumed to follow the chosen distribution type e.g. exponential, normal, lognormal, Gumbel.
- **Prior analysis** - Once the distribution is chosen, a lower and upper bound are then determined - using both objectified and subjective information. The input values of the mean and variance are then derived.
- **Data analysis** - The data sample is directly analysed with a linear regression model using, for example, a weighted least squares approach.
- **Bayesian updating** - Posterior coefficients are calculated, along with the magnitude of confidence in the prior information with respect to the data. The magnitude of confidence is defined by the ratio n/n₀ where almost total confidence is 1/100 and hardly any confidence is 100/1.
- **Posterior analysis** - using the posterior coefficients the parameters of the posterior distribution are calculated - where the resulting distribution functions contains the entire knowledge of the property under consideration.

The foundation uncertainty in the prediction of axial pile capacity calculations is often large because of the fact that the penetration depth, pile length, pile diameter, and ultimate load for the largest piles in the database used to derive the prediction equations, are generally
smaller than those currently used in the North Sea [Lacasse and Nadim, 1996; Senner and Cathie, 1993; Pelletier et al., 1993; and Foray et al., 1993]. Such uncertainty in the model must therefore be evaluated based on comparison pile load tests, deterministic calculations, expert opinions and survey of regulatory organisations, relevant case studies of “prototypes”, results from literature and good engineering judgement [Lacasse and Nadim, 1996; and Wu et al., 1989].

Natural soil characteristics and their physio-mechanical properties were studied in detail in 1997 [Cherubini, 1997]. It was noted that a standard test should be introduced to reduce the variance in data introduced by different test techniques, and that high variability observed for certain parameters (e.g. cohesion) was partially due to “misused statistical procedures in collecting data, not taking into account the variability with depth.”

Bayesian updating has also been applied in the area of optimised inspection planning when knowledge obtained from inspection is used to update the reliability models and optimise future inspection planning accordingly [Onoufriou et al., 1994; Onoufriou, 1996a; Onoufriou, 1996b; Vasudevan and Zintilis, 1994].

4.3 Modelling uncertainty

As mentioned above, model uncertainty arises due to the uncertainty from imperfections and idealisations made in physical model formulations for load and resistance, as well as from choices of probability distribution types for the representation of uncertainty [DNV, 1992]. Model uncertainty occurs as a result of simplifying assumptions, unknown boundary conditions and as a result of the unknown effects of other variables and their interactions that are not included in the structural analysis model [Liu et al., 1996].

Model uncertainty can be described as random factors in a physical model used for representation of load or resistance quantities can, and can be derived by the ratio between the true quantity and the quantity as predicted by the model. A mean value not equal to 1.0 expresses a bias in the model. The standard deviation expresses the variability of the predictions by the model. An assessment of a model uncertainty factor is sometimes obtained through sets of laboratory or field measurements and predictions. However, subjective choices of the distribution of a model uncertainty factor will often be necessary.

Model uncertainty has been described in three main ways: addition of an extra basic variable, multiplication by a coefficient or by the introduction of a random vector.
However, model uncertainty can only be quantified by comparison with other more involved methods that exhibit closer representation of nature or by comparison with collected data from the field or laboratory [Schueremans, 1996].

The first method for describing model uncertainty is by the introduction of an extra basic variable, which is typically assumed to be normally distributed. This can be represented as follows:

\[ g(X,p) + \theta \]

Where: \( g(X,p) \) = limit state, X = variable and \( \theta \) = model uncertainty

This approach was identified in work carried out at Imperial College [Chryssanthopoulos, 1992]. In a study using response surface methodology for probabilistic analysis, Chryssanthopoulos stated that if the true response is unknown, low order polynomials could be used to estimate the response, \( y \), which was denoted as:

\[ y = \hat{y} + \eta \]

Where: \( y \) = ‘true’ response,
\( \hat{y} \) = estimated response
\( \eta \) = error introduced by lack-of-fit approximations.

It was also noted that sometimes, other uncertainties and therefore errors could be introduced, for example, by experimental error if physical experiments have been performed, thus:

\[ y = \hat{y} + \eta + \varepsilon \]

Where: \( \varepsilon \) = error introduced by experimental error

The second method of accounting for modelling uncertainty is by multiplying the model with an unknown coefficient, to be determined from test results. This can often be assumed to be log-normally distributed, and can be represented as follows:

\[ g(X,p)\theta \]

Where: \( g(X,p) \) = limit state, X = variable and \( \theta \) = model uncertainty

This approach was used within the MSL study into jacket and jack-up reliability [MSL Engineering, 1995; and MSL Engineering, 1997].

The third method of accounting for modelling uncertainty is by introducing a random vector field. This is a complex procedure, not commonly adopted, which can be represented as follows [Schueremans, 1996]:
\[ V\{g(X,p)\} \]

Where: \( g(X,p) \) = limit state, \( V \) = random vector and \( \theta \) = model uncertainty.

### 4.3.1 Sensitivity studies

Various sensitivity studies have been performed over the last few years – the background and results to three important studies are presented and discussed in the sections below.

Shell studied sources of reserve strength beyond nominal design strength and showed that actual material strength was a source of uncertainty. As part of their work in investigating implied reliability levels, Shell used a nominal value for the yield strength. It was reported that this yield strength was a conservatively biased estimate of the ‘true’ yield strength, which was expected to be 20% higher for 36ksi steel and 15% higher for 50ksi steel [van de Graaf et al., 1993].

During a reliability analysis, the Shell method takes into account the uncertainty in collapse strength, but it has been found that this is usually of secondary importance to the uncertainty in environmental loading [van de Graaf et al., 1994]. A study undertaken in 1993 performed calculations accounting both for different levels of uncertainty in system strength and for wave crests in the deck. Structural collapse was modelled as either deterministic or probabilistic. In the latter, the distribution of member strength was assumed to take the same form as that of steel yield strength. The lower tail of the distribution was truncated which reflected the rejection of sub-standard material. It was assumed that the material yield strength had a large COV of 10%, and if member strengths are fully correlated, it was assumed that the system collapse strength would also have a COV of 10%. It was concluded that if the material strength and member strengths were statistically independent, the COV of the system strength would reduce. It was also determined that the uncertainty in system behaviour marginally increased the probabilities of failure, which was thought to be due to the uncertainty in load greatly exceeding the uncertainty in strength [van de Graaf et al., 1996].

Another important conclusion from the Shell studies into reliability is that the probability of failure of a structure is largely determined by the return period of the extreme environmental load [van de Graaf et al., 1996].

In the method advocated by DNV using an USFOS/PROBAN approach, uncertainties in the structural capacity model were assumed to be due to the yield stresses and the member
imperfections (magnitude and direction). In the structural loading model, the uncertainties were assumed to be due to the wave height, the thickness of marine growth and the drag and inertia coefficients in Morison’s equation. The wave period and the current speed were assumed to be deterministic values, which were functions of the significant wave height. The ultimate capacity distribution characteristics (i.e. distribution shape and parameters) of a structure were determined by means of Monte Carlo simulation. USFOS was then used for the progressive collapse analysis. Loads due to the structural weight, buoyancy and wind were all assumed to be deterministic. The DNV approach concluded that for offshore structures in general the uncertainties in the prediction of wave forces were greater than the variability in prediction of the system capacity. The prediction of wave loads was subject to uncertainties due to the inherent randomness in the wave process, uncertainties in the seastate parameters and uncertainties in the prediction of wave forces for any given seastate.

In a sensitivity study undertaken by DNV in 1994, [Sigurdsson et al., 1994] it was found that the structural reliability could be estimated without taking into account the randomness in inertia coefficients and marine growth. Further analysis concluded that the inertia coefficient and marine growth could be modelled as deterministic. However, the uncertainty in the drag coefficient should not be ignored.

Uncertainties in the structural model were taken into account in the determination of the cumulative probability distribution of the system capacity by means of a vector of random variables, Z [Sigurdsson et al., 1994]. Uncertainties in the seastate and the wave forces were accounted for in the same manner. Thus, the distribution of the system capacity, \( F_{SC} \), could be represented as:

\[
F_{SC} = F_{SC}(Z_{structure}, Z_{seastate}, Z_{wave-forces})
\]

Further results showed that the distribution of the system capacity was dominated by \( Z_{seastate} \) and hence was sensitive to changes in the annual largest significant wave height. The main conclusion of this work was that despite the fact that there would appear to be large uncertainties in the resistance models, these do not appear to have a large impact on the reliability assessment as this is dominated by the uncertainties in the loading. This is clearly in agreement with the work carried out by Shell [Tromans et al., 1993; van de Graaf et al., 1993] and WSAtkins [Gierlinski et al., 1993].

The reliability analysis used by WSAtkins is based on the first order reliability method (FORM), with random variable probability models used for describing the uncertainty in
basic variables. All of the important environmental parameters, such as wave height, wave period, current speed, drag and inertia coefficients are modelled as explicit random variables. The uncertainty models used for the tensile and compressive yield stresses were the same as in Nordal et al., 1988.

In the structural analysis, member limit-forces in axial tension and bending moments were calculated based on a mean value of yield stress and plastic section properties while the compressive capacity was determined using elasto-plastic buckling analysis. These mean value member capacities were randomised using random multipliers for the reliability analysis to reflect the variability in yield stresses.

The results of a sensitivity study indicated that the system reliability of the jacket structure was strongly influenced by the uncertainty in environmental load parameters [Gierlinski et al., 1993]. A need was identified for more data collection to enable joint probability distribution of all relevant environmental parameters to be developed. In another study carried out by WSAtkins using RASOS in 1994 [Gierlinski, 1994; Gierlinski et al., 1993], it was found that the reliability index was highly sensitive to loading variables, whilst the resistance variables showed very little influence. Of the loading variables, the wave height was found to be the most dominant variable, followed by wave period and then the drag coefficient. It was concluded that the dominance of loading variables introduced high correlation between failure events of different components within a failure path and between failure paths. The correlation coefficients between individual elements were found to be in the order of 0.92-0.95 while the correlation between complete failure paths was as high as 0.98.

Gierlinski [Gierlinski et al., 1993] compared the results from the rigorous RASOS study to those published by Nordal [Nordal et al., 1988], based on a simplified first order reliability method (FORM). The RASOS approach allowed a more rigorous stochastic modelling of the variables and this capacity was used to repeat the analysis, treating wave height, period, current speed and drag coefficient as explicit random variables. The FORM approach used only one basic variable in the modelling of the environmental loading. The member reliability indices were found to be considerably lower in the RASOS approach when compared to corresponding values from the simplified analysis. For the structure studied, it was found that the reliability index was most sensitive to loading variables while the resistance variables showed little influence. Indeed, the combined effect of the loading variables contributed to more than 95% of the total uncertainty. The wave height was the
dominant loading variable, followed by wave period and then drag coefficient [Gierlinski et al., 1993]. This is in agreement with work carried out by DNV/SINTEF [Sigurdsson et al., 1994] where it was concluded that the wave height was the most important variable.

It is important to note that none of the studies described above included foundation uncertainties. However, if this had been included this uncertainty may be comparable to the loading uncertainties and would become significant and perhaps dominant in the overall uncertainty of the system.

4.3.2 Generic uncertainty and sensitivity considerations

Probability has the appearance of precision because it is a mathematical quantity. It derives the stochastic nature of the frequency of the occurrence of events and, given sufficient failure data, a ‘classic’ probability may be calculated to reflect the likelihood of a particular event occurring. If insufficient failure data are available, the probability can be calculated from a combination of intuitive or subjective assumptions combined with a set of observation data. The Bayesian technique, which uses prior knowledge (e.g. failure rates in similar but not identical circumstances elsewhere, combined with the views of experts) redefines the probability steadily, as information that is more specific becomes available [see Luxhoj, 1997].

As far back as 1978, DNV noted that “probabilistic reliability is the only meaningful concept that can be used to obtain a logical and objective distribution of risks and safety requirements” [Fjeld, 1978]. Soares et al in 1995 perceived that “by formulation of the limit state equations in terms of parameters which may be estimated or even observed by inspections and experiments and by representation of these parameters in terms of stochastic processes and/or variables, the probability of occurrence of any considered combination of structural states may be estimated” [Soares et al., 1995]. For more information on limit states, see [Vrouwenvelder, 1995].

From the literature studied, it has been seen that it is generally accepted that the environmental parameters are those having the most significant effect on reliability analysis results. Uncertainty in loading appears to account for a significant proportion of the total uncertainty. Foundation uncertainty becomes a significant parameter where it dominates the resistance. It is generally accepted that the uncertainty in prediction of foundation axial pile capacity calculations is often large [Lacasse and Nadim, 1996; Senner and Cathie, 1993; Pelletier et al., 1993; Foray et al., 1993], and that an estimate of such uncertainty should be
evaluated on the basis of a combination of pile tests, calculations, opinions and good engineering judgement [Lacasse and Nadim, 1996; Wu et al., 1989]. It has also been concluded that variance is induced in foundation data due to different test techniques [Cherubini, 1997].
5. Structural Models

5.1 Advancement of computer models

A significant amount of effort has been expended on the development of computer models that represent structures. Models have been developed using general-purpose commercial software programs [Orisamolu et al., 1994], but recent developments have been to develop specialised software. These can deal more efficiently and effectively with large models and with large displacements, and have been used for analysis of offshore structures [e.g. Gierlinski, 1994; BOMEL, 1992; also Bolt et al., 1995].

In 1992, a description of system reliability formulations was presented [Moses and Liu, 1992]. After some initial studies on idealised behaviour of structures, they concluded that the three most important factors in the assessment of structural models were

- a realistic structure geometry,
- a range of material behaviours to describe both brittle and ductile situations,
- use of basic variables to assess their relative interdependence.

In 1994, a European initiative was launched on “evaluation of technical models used within the major industrial hazard areas”. It was reported that the essential requirements of models were [Model Evaluation Group, 1994]:

- clear objectives and performance standards
- full and open documentation including illustrative example problems
- unambiguous version control
- openly reported performance evaluation.

A model evaluation protocol was developed which was aimed at providing guidance on how to evaluate a technical model. The guidelines were intended to be viewed as minimum requirements [see Model Evaluation Group, 1994b].

A number of detailed structural models have been developed and analysed over the past few years. Detailed descriptions of different non-linear software were included in a review of the ultimate strength of tubular framed structures which was carried out in 1992 and updated in 1995 [Bolt et al., 1995]. Aspects addressed included information on beam element formulations, spring elements, method of solution, load application, pre-processing/model
creation facilities, offshore code-checking/post-processing facilities, results/output facilities, ongoing developments and date of development with current status. Details of analytical investigations published in technical literature were also included [Hamilton and Murff, 1995].

5.2 Specific examples of models and analyses undertaken using different software

The following section describes models for some of the key structures analysed, based on information provided directly by sponsors as well as from literature available in the public domain.

5.2.1 Analyses undertaken using ABAQUS

In 1993 Brown and Root Limited undertook an assessment of the redundancy and target reliability of the Lomond platform for Amoco (UK) Exploration Company [Brown and Root, 1993b]. The general purpose FE software package ABAQUS was used for the non-linear analysis, with PATRAN being used as the graphical pre- and post- processor package. In this analysis, the complete jacket structure with a simplified version of the topside was modelled in which ABAQUS was used to include both geometric and material non-linearity. A finer mesh was used for the jacket, whilst a coarser mesh was used in the region of the topside. The wave loadings were applied as a set of nodal loads at all the nodes on each of the horizontal bracing levels and were not represented as distributed loads as had been applied in a previous analysis using an in-house Brown and Root software package called DAMS.

This study looked into the effect of redundancy and target reliability within a strategy for optimised inspection planning. In total, 12 analyses were performed on a combination of intact and damaged structures, with fixed, fatigue springs or 50% fatigue springs supports. Every structural member was modelled with several three-node finite-element beam elements, ensuring that their length did not exceed 5.0m. The elements used to model the topside were generally larger than those in the jacket. There was no explicit modelling of the conductors, but their presence was accounted for by the enhancement of wave loads on their supporting nodes. All risers, J-tubes and their supports were fully modelled. Local thickening of the legs at can locations was modelled, but member offsets were not modelled because numerical errors in the analysis might have been introduced. Modelling of member imperfections was not included since ABAQUS could not allow for implementation of
imperfections directly and additional nodes would have had to be included. Such additional nodes were only deemed necessary for structures with a large slenderness, which was not the case with the Lomond jacket. The environmental storm condition that was applied to the platform from the Westerly direction [Brown and Root, 1993b; Brown and Root, 1994; and Brown and Root, 1995] is described in Table 2.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wave height, H</td>
<td>18.6m</td>
</tr>
<tr>
<td>Wave period, T</td>
<td>15.1s</td>
</tr>
<tr>
<td>Surface current</td>
<td>0.44m/s</td>
</tr>
</tbody>
</table>

Table 2: 50-year extreme storm condition for Lomond platform for W direction [Brown and Root, 1993b]

The different conditions and scenarios analysed are described in the table overleaf.

<table>
<thead>
<tr>
<th>Waves</th>
<th>Scenarios studied</th>
<th>Effects examined</th>
</tr>
</thead>
<tbody>
<tr>
<td>W (critical)</td>
<td>• Fatigue springs support - intact &amp; 7 damaged</td>
<td>• Environmental load factor</td>
</tr>
<tr>
<td></td>
<td>• Fixed support - intact</td>
<td>• Redundancy</td>
</tr>
<tr>
<td></td>
<td>• 50% fatigue springs support - intact &amp; 2 damaged</td>
<td>• Target reliability</td>
</tr>
</tbody>
</table>

Table 3: Conditions studied for the Lomond platform

Pile supports for the jacket were modelled as a number of springs; the stiffness of which was taken from those used in an earlier fatigue analysis. Six linear springs were placed at each leg. A study into the support restraint showed that there was a 2.5% reduction in the load factor for the transition from a fixed to a flexible support [Brown and Root, 1993b].

Initially all constant loads were applied with a load factor, \( \lambda \), of unity. The environmental loads were then applied incrementally until structural instability caused non-convergence of the forces/moments at nodes in the solution algorithm. The plateau on the load-deflection curve determined the ultimate capacity of the structure.

In a later study conducted on the Lomond structure [Brown and Root, 1994], modelling of the foundation was included with non-linear springs being utilised to represent the soil and an allowance for pile group interaction was made. The pile group interaction was developed using a secondary program, MINDLIN, to calculate the interaction effect. Additional displacements were applied directly to the ABAQUS model by varying the boundary conditions on the springs throughout the analysis. Both the intact and the worst case
damaged structure were studied. From examination of the reserve strength ratios, it was found that the pile-group interaction effects were negligible for this platform.

Another investigation undertaken on Lomond [Brown and Root, 1995], studied the effect of including the conductors in the model. It was concluded that conductor stiffness contribution amounts to approximately 2% of the reserve strength load factor.

5.2.2 Analyses undertaken using USFOS

Shell has studied several of its platforms in detail [Tromans et al., 1993; van de Graaf et al., 1993]. The Tern platform, designed mid 1980s, made full use of structural analysis, and was a fully optimised design. The Inde-K platform, designed in 1971, is one of Shell’s older platforms, which was designed using a conventional approach. The Inde-K structure is a manned, self-contained drilling production platform that was the subject of a detailed reassessment in 1991 Si Boom et al., 1993]. The original design and the reassessment analyses were studied with different environmental conditions. The wave height used was 14.5 m and 15.8 m and the wave period was 12.3 s and 12.7 s for the original design and the reassessment models respectively.

In a study undertaken in 1994, DNV/SINTEF used the program USFOS. Two models, A and B, were studied by DNV/SINTEF [Sigurdsson et al., 1994] using a combination of the two computer packages, USFOS, which is a specialised non linear structural collapse analysis, and PROBAN (PROBabilistic ANalysis) which is a dedicated probabilistic program. It is worth noting that a review of different reliability analysis software packages was undertaken in 1994 in Canada [Orisamolu et al., 1994]. It was shown that PROBAN had a number of unique features which enabled it to enhance computational efficiency. In addition, it also had the capability for computing parametric sensitivity and importance factors for components and systems. Model A had 4 legs, and was installed in 70 m water depth, whilst model B had 8 legs and was installed in 77 m water depth. The two models were studied with different environmental conditions. The wave heights used were 29 m and 12.2 m, the wave periods were 17.5 s and 11.7 s, and the current speeds were 1.25 m/s and 1.0 m/s for models A and B respectively.

It was reported that “the basic principle behind USFOS is to represent each individual member in the structure by one finite element. This is allowed for by using the exact solution to the differential equations for a beam subjected to end forces as shape function to
the elastic displacements, which take into account large lateral rotations at the element. Closed form solutions are obtained for the elastic total and incremental stiffness matrices which contain all information required to identify buckling of members or sub-systems.”

USFOS has the capability of modelling non-linear behaviour. This is done by means of plastic hinge theory in which the yield criterion is expressed in terms of two plastic interaction functions; one represents first fibre yield and the other represents full plastification of the cross-section.

In creating the models, simplified representations of the original structure were used. The structural elements which did not contribute to the load carrying capacity, were not included in the computer model. The topside loads and wind loads were applied as concentrated forces at the four topside corners. In the DNV/SINTEF investigation into whether the system capacity could be directly related to the total base shear force, a deterministic model of the structure was used, in which all uncertainties were assumed to be on the load side. For a given seastate, the wave height and period were kept fixed, and for this investigation only one wave direction was considered.

5.2.3 Analyses undertaken using RASOS

An eight-legged X-braced jacket structure has been studied extensively by WSAtkins [Gierlinski et al., 1993], using the program RASOS (Reliability Analysis System for Offshore Structures). This structure was studied in both the intact state and the damaged state. The damaged state was introduced by including a severe imperfection (0.01 x member length) into one of the braces.

The eight-legged structure was analysed with members being modelled using two-node beam column elements. Only the jacket was analysed, so the topside and foundations were not modelled. The outer horizontal members in the upper most horizontal framing of the structure were assigned artificially high values of Young’s modulus in order to model the stiffness of the deck. The remaining structural material had the characteristic values:

- Young’s Modulus = 210 GPa
- Poisson ratio = 0.3
- Density = 7850 Kg/m³
- Yield stress = 300 MPa.
Leg and bracing members that carry high loads in both axial and transverse directions were modelled as non-linear beam columns. These beam columns were therefore allowed to develop plastic hinges, either by yielding in tension or by buckling in compression [Gierlinski et al., 1993].

In early 1997, WSAtkins undertook a study into the structural system reliability of Lomond [WSAtkins, 1997a] using RASOS. A number of structural analyses were undertaken including static, linear elastic, component utilisation and non-linear progressive collapse analysis. System reliability analysis was performed by taking into account the uncertainties in both the loading and resistance parameters, for both intact and postulated damage scenarios. The structural model consisted of three types of load bearing components: leg and tubular members, piles and joints. Secondary components including risers, J-tubes and conductors were modelled as tubular elements and were used only for the environmental load generation. The inherent stiffness of these secondary components was not included in any of the response calculations.

In this study, the pushover capacity of the jacket was evaluated under both the design and the extreme environmental loading conditions. For the latter, wave-in-deck forces were taken into account. Progressive collapse analyses were undertaken for several values of wave height. For each wave height, a crest position was established which corresponded to the maximum value of the total base shear and the associated response of the structure was recorded.

5.2.4 Analyses undertaken using SAFJAC

Detailed examination of the Montrose jacket was undertaken by Billington-Osborne Moss Engineering Limited (BOMEL) from 1994 to 1997 [BOMEL, 1996a; BOMEL, 1996b] for Amoco (UK) Exploration Company. The reserve strength of the platform was the focus for the study. In particular the reserve strength observed under the 50-year extreme storm conditions. Such conditions are described in the table below, and are shown for the Northerly direction.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wave height, H</td>
<td>77.4 feet (23.6m)</td>
</tr>
<tr>
<td>Wave period, T</td>
<td>15.6 sec</td>
</tr>
<tr>
<td>Surface current</td>
<td>3.99 feet/sec (1.22 m/sec)</td>
</tr>
</tbody>
</table>

Table 4: 50-year extreme storm conditions for Montrose platform Northerly direction
BOMEL studied three different wave directions and four different jacket conditions in order to examine the effects of wave direction, foundation stiffness and joint flexibility. These conditions are described in the following table.

<table>
<thead>
<tr>
<th>Waves</th>
<th>Scenarios studied</th>
<th>Effects examined</th>
</tr>
</thead>
<tbody>
<tr>
<td>True N NW SW</td>
<td>• Fixed piles &amp; rigid joints</td>
<td>• Wave direction</td>
</tr>
<tr>
<td></td>
<td>• Piled foundations &amp; rigid joints</td>
<td>• Foundation stiffness</td>
</tr>
<tr>
<td></td>
<td>• Piled foundations &amp; joint flexibility &amp; strength for highly stressed joints</td>
<td>• Joint flexibility</td>
</tr>
<tr>
<td></td>
<td>• Piled foundations &amp; joint flexibility &amp; strength for highly stressed joints with 8 members removed due to low fatigue lives arising from weld defects</td>
<td>• Safety critical elements</td>
</tr>
</tbody>
</table>

Table 5: Conditions studied for the Montrose platform

The pushover analyses conducted for the Montrose platform were performed using BOMEL’s software SAFJAC. The loading for the pushover analyses was applied in two stages: the first was the still water or operating conditions of the platform and the second stage was the extreme storm conditions. The extreme storm conditions were increased proportionally to the design level load factor ($\lambda_p = 1.0$) and then beyond in order to determine the ultimate load factor ($\lambda_{p_{max}}$).

The ultimate load factor ($\lambda_{p_{max}}$) results from the pushover analyses are shown in the following table for the true North direction for the Montrose platform.

<table>
<thead>
<tr>
<th>Scenario</th>
<th>Ultimate load factor, $\lambda_{p_{max}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fixed piles &amp; rigid joints</td>
<td>3.65</td>
</tr>
<tr>
<td>Piled foundations &amp; rigid joints</td>
<td>2.66</td>
</tr>
<tr>
<td>Piled foundations &amp; joint flexibility &amp; strength for highly stressed joints</td>
<td>2.66</td>
</tr>
<tr>
<td>Piled foundations &amp; joint flexibility &amp; strength for highly stressed joints &amp; 8 member removed due to low fatigue lives arising from weld defects</td>
<td>2.55</td>
</tr>
</tbody>
</table>

Table 6: Results obtained for the Montrose Platform

The main conclusion drawn from the study of Montrose was that the axial pile and end bearing capacities of the soil were the main controlling factors on the reserve strength of the foundations and hence on the overall jacket. From the results above, it can clearly be seen that varying the joint rigidity has no significant effect on the reserve strength, because the
foundations are the dominant feature. The removal of eight of the members because of observed weld defects had little effect and only reduced the ultimate load factor by less than 5%. The pushover analyses also allowed the safety critical members to be identified - these were the diagonal members in rows 2 and 3 above the bottom bay and again, the foundation was considered to be a safety critical element.
6. Environmental parameters

Structural reliability analyses of offshore structures are dependent upon the environmental loads assumed to be acting on the structure. Loads induced by waves, current and wind are all of a random nature and are usually described by probability theory. In the past, the conventional practice adopted for the treatment of waves, current and winds forces was to treat each factor separately and then combine the independent extremes simultaneously. However, this has been found to be over conservative and can result in an overestimation of the design loads required. More recently, the development of more reliable databases of hindcast data [e.g. Grosskopf et al, 1991; Peters et al., 1993] has enabled a joint description of these quantities to be determined [Onoufriou et al., 1997; Prior-Jones and Beiboer, 1990].

The UK energy industry is seen as one of the foremost in the collection of meteorological and oceanographic data. Many studies have been undertaken on the overall approach to collecting, assessing and analysing the environmental conditions encountered offshore [e.g. Galano et al., 1995; Murray et al., 1995; Grant et al., 1995; Sharma and Grosskopf, 1994; Ye and Zhang, 1994; Bea and Craig, 1993b; Jonathan et al., 1994; Taylor, 1992; WSAtkins, 1995; Vugts, 1990; Vinje and Haver, 1994; Karunakaran et al., 1994]. These studies present important information for the development of methods for the interpretation and calculation of loads and responses of offshore structures. An important study in this area was commissioned by HSE and undertaken in 1993/4 by Metocean Plc which presented several data sets of wind and wave data for all areas of the North Sea including the South West approaches and West of Shetland [Metocean Plc, 1994].

A joint industry project investigation into the analysis of wave loads seen on Shell’s Tern platform in the Northern North Sea 1990-1992 was undertaken by WSAtkins in 1995. During this period, 15 storms were experienced with significant wave height above eight metres. Comparisons were made between different techniques for predicting the wave height, and the results were then compared to actual wave measurement [WSAtkins, 1995]. It was found that the measured water velocities corresponded closely with a Stokes wave, and that the effect of current on the peak-to-peak wave loads considered was small.
6.1 Waves

Ocean waves generally refer to the moving succession of irregular crests and hollows on the ocean surface. They are generated primarily by the drag of the wind on the water surface and hence are greatest at any offshore site when storm conditions exist there.

In general, only one or two wave approaches are used in a structural platform analysis. However, it is important to note that for a full analysis more damage scenarios considering a number of wave approach directions should be carried out [Bitner-Gergusen and Soares, 1997].

6.1.1 Wave theories

It is customary to analyse the effects of surface waves on structures either as a single wave chosen to represent extreme storm conditions in the area of interest, or by statistical representation of the waves during extreme storm conditions [Castillo et al., 1995; Soares and Ferreira, 1995; Labeyrie and Schoefs, 1995]. In either case, it is necessary to relate the surface-wave data to the water velocity, acceleration, and pressure beneath the waves [Dawson, 1993]. This can be achieved by the application of an appropriate wave theory – the most widely used approach being to use either the Airy [Gierlinski et al., 1993; Olufsen and Bea, 1989] or the Stokes [Tromans et al., 1993; van de Graaf et al., 1993; Efthymiou and Graham, 1990] wave theory. A more recent development, the NewWave theory, has also been used [Tromans et al., 1993; van de Graaf et al., 1993].

The recommendations within API RP2A for the 2-D wave kinematics [API , 1993a] include guidance on the regions of applicability of stream functions, Stokes V, and linear/Airy wave theory, dependent upon the occurrence of shallow or deep water waves.

A relatively simple theory of wave motion was developed by Airy in 1842. This description assumes a sinusoidal waveform whose height is small in comparison to the wave length and the water depth. The Airy theory can be used for preliminary calculations and for revealing basic characteristics of wave-induced water motion. It also provides a basis for the statistical representation of waves and induced water motion experienced during storm conditions.
An example of the application of the Airy wave theory is in a study performed by WSAtkins, in which water particle velocities and accelerations were separated into a random component and a deterministic component [Gierlinski et al., 1993].

An extension of the Airy theory to waves of finite height was formulated by Stokes in 1847. This method involved the expansion of the wave solution in series form and a determination of the coefficients of the individual terms in order to satisfy the appropriate hydrodynamic equations for finite-amplitude waves. Stokes carried this analysis forward to third order of accuracy in the wave steepness. An extension of this method to fifth order was accomplished by Skejallbreia and Hendrickson in 1961. This work, commonly referred to as the Stokes fifth-order wave theory, has been widely employed in offshore engineering calculations for finite amplitude waves with lengths less than about 10 times the water depth.

A more accurate and realistic hydrodynamic model was derived by Shell, which more realistically describes the kinematics associated with extreme crests [Tromans et al., 1993; van de Graaf et al., 1993]. This NewWave method included all spectral and directional properties of real storm waves. Statistics were performed for whole storms rather than just the standard 3-hour intervals, which allowed a more detailed investigation to be undertaken. The North European Storm Study (NESS) data was used for the hindcast database. The symbol X was used to represent a variable that may be crest elevation, wave height, or a global load (such as base shear). The cumulative probability of the extreme value of X in a storm, characterised by a most probable extreme value, Xmp, is then P(X|Xmp). The probability density function of Xmp was therefore p(Xmp). The probability distribution of the extreme value of X in any random storm (r.s.) of unknown Xmp is:

\[
P(X|r.s) = \int P(X|Xmp).p(Xmp)\,dXmp.
\]

The probability distribution of the extreme value of X in some long time interval (say 100-years) was therefore:

\[
P(X|nT) = (P(X|r.s.))^nT
\]

Where: \( n \) = average rate of storms per year.

In NewWave, the definition of the surface elevation, \( \eta \), in the region of the wave crest was:

\[
\eta = A \cdot \left(16/H_s^2\right) \int S(\omega) \cdot \cos(k(\omega)x - \omega t) \, d\omega
\]

Where: \( A = \) crest elevation

\( S(\omega) = \) ocean surface energy spectrum
\( H_s \) is four times the standard deviation of ocean surface elevation.

It should be noted that in traditional periodic wave theories, e.g. Stokes, the fact that large waves occur when, by chance, many waves of different speeds and directions come into phase is not captured. In NewWave, however, the phases and amplitudes of the components are selected according to statistical theory to capture the most probable extreme wave and its water particle kinematics. It is this, therefore, that enabled the NewWave methodology to describe more accurately the wave model.

6.1.2 Wave forces

The force exerted on a fixed vertical pile by surface waves was first considered by Morison in 1950. This theory is restricted to conditions where the diameter of the pile was small in comparison to the length of the waves encountered, so that the distortion of the waves by piles is negligible [Dawson, 1993]. For the analysis of wave forces on piles, this approach considers that the total force is due to an inertial force component arising from the water particle accelerations and a drag component due to friction and boundary layer effects [Heideman and Weaver, 1992].

Loads on a structure are widely calculated by the use of the Morison’s formula, as follows:

\[
F = 0.5 \rho C_D D |u| u + 0.25 \rho \pi D^2 C_M a
\]

Where: 
- \( F \) = force per cylinder length in flow and motion direction
- \( \rho \) = density of seawater
- \( C_D \) = drag coefficient
- \( D \) = cylinder diameter
- \( u \) = water particle velocity
- \( C_M \) = inertia coefficient
- \( a \) = water particle acceleration

The first term on the right hand side of this equation is referred to as the drag term and is seen to be proportional to the square of the water velocity. The absolute value sign is used to ensure that the sign of the drag component will coincide with that of the velocity. The second term is referred to as the inertia term and is seen to be proportional to the water acceleration.

The API RP2A recommendations advise the use of Morison’s equation. The computation of the forces exerted by waves on a cylindrical object dependent on the ratio of the wavelength
to the member diameter. API states that when this ratio is large, i.e. greater than five, the member does not significantly modify the incident wave. Thus, the wave force can then be computed by the sum of the drag and inertia forces as described in Morison’s equation.

The values of drag and inertia coefficients vary with the maximum water velocity, of the wave motion and with the wave period, through the dimensionless numbers known as the Reynolds number and the Keulegan-Carpenter number. The Reynolds number is representative of the effect of viscosity, while the Keulegan-Carpenter number is representative of the effect of the wave period. In addition, both the drag and inertia coefficients can also be affected by member roughness [Dawson, 1993]. Limited experimental data exist on the variation of drag and inertia coefficients with these numbers and hence it is usual to assume that they are both constants. The values of the inertia and drag coefficients are of particular significance for both the design and reassessment of offshore structures and are still subject to discussions and ongoing assessments. Within the offshore industry, values for the drag coefficient are usually within the range 0.6 to 1.0, while values for the inertia coefficient are usually within the range 1.5 to 2.0 [API, 1993a; see also Digre et al., 1994].

In API RP2A (1993) it is stated that for “typical” design situations, the global platform wave forces can be calculated using the following values for unshielded circular cylinders, where there is a steady current with negligible waves, or for the case of large waves:

- Drag coefficient: smooth $C_d = 0.65$, rough $C_d = 1.05$
- Inertia coefficient: smooth $C_m = 1.6$, rough $C_m = 1.2$

For smooth members a drag coefficient of 0.64 was used and for rough members a drag coefficient of 1.2 used for a Shell study of a North Sea platform [Tromans et al., 1993; van de Graaf et al., 1993]. A similar value was applied by WSAtkins, where a drag coefficient of 0.7 was applied with a lognormal distribution, as an explicit random variable [Gierlinski et al., 1993]. This is in agreement with work carried out by DNV, where it was found that when reliability was dominated by uncertainty in seastate, and in particular when drag was especially important, the drag coefficient could be modelled using a lognormal distribution [Sigurdsson et al., 1994].

For a smooth member the inertia coefficient, $C_M$ was taken as 2.0 and for a rough member the inertia coefficient was taken as 1.5 in a Shell study [Tromans et al., 1993; van de Graaf et al., 1993]. Similarly, a value of 1.8 for inertia coefficient was used by WSAtkins.
[Gierlinski et al., 1993], which was modelled as an explicit random variable. Work by DNV agrees with this approach, and in a study carried out the inertia coefficient was applied as a lognormal distribution in those cases where inertia was important. However, it was concluded that the inertia coefficient could be modelled as a deterministic value in those instances where inertia was less critical [Sigurdsson et al., 1994].

When an overall wave/current load profile is determined, the total wave force per unit length perpendicular to a structural member can be estimated using Morison’s equation [Gierlinski et al., 1993; Sigurdsson et al., 1994]. Water particle velocity and current velocity, both of which are assumed time independent, can then be estimated perpendicular to the structural member [Sigurdsson et al., 1994]. These may have been derived using the Airy wave theory [Gierlinski et al., 1993], where water particle velocities and accelerations were subsequently separated into random and deterministic components. These were later transformed to the global co-ordinate systems and expressed as equivalent member or nodal loads. The final vector of distributed forces was then expressed in terms of eight deterministic force vectors, each multiplied by a random factor. The deterministic components were a function of member diameter and structural topology, whilst random multipliers were functions of uncertain environmental parameters.

The API wave load application procedure can be summarised in the following figure. The background to the static wave force procedure for platform design was reported in a significant paper [Heideman and Weaver, 1992].

![Figure 2: Diagram showing the API procedure for calculation of wave plus current forces for static analysis [API, 1993b]](image-url)

6.2 Current
Currents at a particular site can contribute significantly to the total forces exerted on the submerged parts of an offshore structure. Currents refer generally to the motion of water that arises from sources other than surface waves. Tidal currents, for example, arise from astronomical forces, whereas wind-drift currents arise from the drag of local wind on the water surface. During storm conditions, currents at the surface of 0.6 m/s or more are not uncommon, giving rise to horizontal structural forces that equal 10% or more of the wave-induced forces [Dawson, 1993].

According to API RP2A, the current speed near the platform is reduced from the specified “free stream” value by blockage. This means that the presence of the structure causes the incident flow to diverge; some of the incident flow goes round the structure rather than through it, and the current speed within the structure is reduced.

Whether the current is important in modelling extreme environmental loading, depends on the location of the structure as well as the magnitude of the current. The effective current through the platform can be modelled as the equivalent of the undisturbed current divided by \([1+(\text{hydrodynamic area}/4 \times \text{frontal area})]\) [Taylor, 1991]. This was used by Shell in conjunction with the NewWave method [Tromans et al., 1993; van de Graaf et al., 1993].

In a study where structural analyses were performed on North Sea platforms with and without current, significant differences in the structural response were noted but the system capacity was found to be almost the same [Sigurdsson et al., 1994]. This means that in this particular case, current in the overall system capacity could be estimated without taking into account the uncertainty of the current loading pattern. However, in a study that utilised the response surface technique, WSAtkins modelled the current speed as an explicit random variable [Gierlinski et al., 1993].

### 6.3 Wind

Over-water wind during storm conditions is significant in the design of offshore structures because of the large forces it can induce on the upper exposed parts of the structure. The forces exerted on a structure by wind depend on the size and shape of the structural members in the path of the wind and on the speed at which the wind is approaching. The greatest wind speed to be expected at a particular site can be estimated from analysis of daily weather records if available. Due to wind fluctuations over the measurement time,
such records necessarily contain averaged wind-speed measurements over a finite interval of time [Dawson, 1993].

The wind force acting on an offshore structure is the sum of the wind forces acting on its individual parts. For any part, such as a structural member or deck, the wind force arises from the viscous drag of the air on the body and from the difference in pressure on the windward and leeward sides. In fixed offshore structures, the wind load can be modelled as a quasi-static load process, or as a deterministic quantity [Gierlinski et al., 1993; Sigurdsson et al., 1994]. A typical value for the static wind speed for a North Sea structure is approximately 50 m/s [Sigurdsson et al., 1994].

The API RP2A recommendations for wind forces note that wind loads are dynamic in nature, but that some structures will respond to them in a nearly static fashion. For conventional fixed steel templates in relatively shallow water, winds can be a minor contributor to global loads, typically less than 10%. However, in deeper water, wind loads can be significant and should be studied in detail with attention being paid to the mean profile, gust factor and turbulence intensity [API, 1993a].

The wind force on an object according to API should be calculated by using the following:

\[ F = \left( \frac{\rho}{2} \right) \cdot V^2 \cdot C_S \cdot A \]

Where: 
- \( F \) = wind force
- \( \rho \) = mass density of air (at standard temperature and pressure)
- \( V \) = wind speed
- \( C_S \) = shape coefficient
- \( A \) = area of object

### 6.4 Extreme environmental event methodologies

The design of an offshore structure is largely governed by the severe environmental loadings exerted on the installation, where such loadings arise from extreme storm conditions. Design storms are widely chosen as extreme metocean conditions that have a specific recurrence interval of say 50 or 100 years. Suitable wind and wave conditions are thereby predicted, along with estimates of the storm tides. The rise in the water level experienced during a storm result from astronomical tides and storm surges. Current conditions must also be appraised by a study of the local conditions and the water velocities associated with the storm current must be added to those caused by the wave motion. The 50 or 100-year response based design condition, therefore, is defined as the environment that generates the
50 or 100-year responses in the generic structure. This environmental estimation incorporates assessments of wind, waves, tides, surges and currents. For further information on the use of response based design see [Huyse et al., 1995].

In the past, the traditional method for prediction of the 50-year wave was performed by working out the proportion of the probability, P, that the wave height was greater than the significant wave height, \( H_s \), i.e. \( P(H_s < H_s') \) for the measured data below a threshold \( H_s' \). \( H_s' \) was then raised from its minimum value in steps, usually of 0.5m.\( P(H_s < H_s') \) and then plotted against \( H_s' \) using scales which would give a straight line if the assumed probability law was obeyed. It was found that for UK waters, the Weibull formula gave the best-fit [Sigurdsson et al., 1994]. The line was then extrapolated to the probability corresponding to one exceedance in 50-years, thus giving \( H_{50} \). In the 1970s Battjes proposed a method in which the distribution was estimated from measured three-hourly wave height and zero crossings. In 1978, a modified Battjes technique was developed [Tucker, 1989], which aimed to calculate the expected number of waves exceeding a height \( h \) in a year, rather than the probability of a randomly-chosen wave height exceeding \( h \) [Sigurdsson et al., 1994].

A probabilistic environmental model was developed by Haver [Vinje and Haver, 1994], which was based on the annual distribution of the extreme load on the structure, \( F_{L\text{ annual}} \), that was represented as:

\[
F_{L\text{ annual}} = F_{L\text{ annual}}(Z_{\text{seastate}}, Z_{\text{wave-forces}})
\]

Where: 
- \( Z_{\text{seastate}} = \) vector of random variables of the seastate 
- \( Z_{\text{wave-forces}} = \) vector of random variables of the wave-forces.

On this basis, the distribution of the system capacity of the structure, \( F_{SC} \), was represented:

\[
F_{SC} = F_{SC}(Z_{\text{structure}}, Z_{\text{seastate}}, Z_{\text{wave-forces}})
\]

Where: \( Z_{\text{structure}} = \) vector of random variables of the structure.

An example of the use of 100-year environmental parameters was in a study by WSAtkins [Gierlinski et al., 1993]. The 100-year environmental parameters were represented by an explicit analytical function for member internal forces and the random response was defined as a summation of the response to 8 deterministic load vectors, factored by 8 corresponding random multipliers.

The API RP2A recommendations state that the extreme wind, wave and current load is the force applied to the structure due to the combined action of the extreme wave (typically 100-
year return period) and associated current and wind, accounting for the joint probability of occurrence of winds, waves and currents (both magnitude and direction) [API, 1993a].

As previously mentioned, the loads induced by extreme storms are critical in the design of offshore structures for location in severe seas. The load arises from a combination of waves, currents and winds, though waves are generally the most dominant factor [Swan, 1992]. It has been widely practised to conservatively assume that the 100-year wave, the 100-year wind and the 100-year current occur simultaneously, acting in the same direction. For a typical jacket structure, this will lead to the derivation of a 100 year “design” load which is significantly more severe than the “true” 100 year load. This traditional practice is conservative in two ways: extremes do not necessarily occur simultaneously and extremes will not necessarily combine in the worst possible way [Wen and Banon, 1991; Spronson, 1996; Prior-Jones and Beiboer, 1990].

More recently, work has been focused on investigating new methods, which can account for the joint probability of occurrence of the winds, waves and currents.

6.4.1 NOCDAP, 1985

The Norwegian Ocean Current Data Analysis Programme (NOCDAP) was undertaken in 1985 as a joint venture between Esso (Norway) and Conoco (Norway). In one study, as part of this programme, simultaneous wind, wave and current data during 21 storms, spanning four winters at one location in the Northern North Sea, were analysed in order to assess joint probabilities of occurrence [Gordon et al., 1985]. This work looked into developing a new procedure for describing the joint probability of occurrence, but concluded that there were still some significant aspects that required further investigation.

6.4.2 Metocean Plc, 1990

In an extensive review into the use of joint probability in deriving environmental design criteria carried out in 1990 [Prior-Jones and Beiboer, 1990] Metocean Consultancy Ltd (now Metocean Plc) investigated the then current approaches to joint probability concepts. They reported that although the need for quantifying the joint occurrence of wind, waves and currents was more important from an engineering point of view, it was more difficult to quantify this than the joint probability of tide and surge.
Various studies were reported which had undertaken investigations to quantify the effect of applying the conservative 50yr. wave + 50yr. current + 50yr. wind approach. These results are summarised in Table 7.

<table>
<thead>
<tr>
<th>Author</th>
<th>Overestimate of base shear</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prior-Jones et al. (1983)</td>
<td>25% +</td>
</tr>
<tr>
<td>Nielsen et al. (1986)</td>
<td>West Sole field 22% - 34%, Gorm field 4-12%</td>
</tr>
<tr>
<td>Madsen et al. (1988)</td>
<td>20%</td>
</tr>
<tr>
<td>Haver &amp; Winterstein (1990)</td>
<td>15% - 20%</td>
</tr>
</tbody>
</table>

Table 7: Estimates of the effect on base shear of applying the 50yr. wave + 50yr. current + 50yr. wind approach compared to applying the joint probability of occurrence approach [Prior-Jones and Beiboer, 1990]

6.4.3 Wen and Banon, 1991

Wen and Banon undertook a study into the combination of wind, wave and current loads in the Gulf of Mexico, with particular emphasis on hurricane load combination criteria, for the API technical advisory committee 88-20 [Wen and Banon, 1991]. This work used the fact that the commonly used conservative procedure was based on a “worst case” scenario in which the direction of lowest resistance of a structure coincided with that of the highest force from a hurricane. This resulted in an upper bound estimate for the risk, but the degree of conservatism remained unknown. In this work, the hurricane event directionality was explicitly included in the hurricane models. By using a Monte Carlo simulation approach, it was found that consideration of the asymmetry in the platform loading and resistance could lower the risk by a factor of 2 to 4 [Wen and Banon, 1991]. It was also concluded that if the strong axis of the platform was aligned with the predominating direction of hurricane waves, the platform probability of failure could also be reduced by a similar factor.

6.4.4 Shell, 1994-6

A method to obtain a more accurate prediction of joint met-ocean conditions was developed by Shell in 1995. This method used the most probable extreme individual wave of the storm history rather than the peak significant wave height. The most probable extreme individual wave height was defined as a function of several of the most severe seastates of the storm, and hence, as this method used more data, it was found to be less sensitive to “noise” [Tromans and Vanderschuren, 1995].

Shell have developed their own techniques to establish the “long term distribution of environmental loading and for back calculating joint metocean conditions for a specified
return period” [Efthymiou et al., 1997]. Such methods were developed in order to account for the statistical distribution of wave height within successive sea states of a storm. These methods enabled all sources of environmental variability to be accounted for and established the long time scale and the joint occurrence of winds, waves and currents.

The long term load distributions derived following the Shell methodology were re-stated in terms of more commonly used probability distributions, namely by developing a lognormal approximation to describe the 20-year and 100-year load. This takes into account the joint probability of waves, currents and winds in their definition. Lognormal long term loading distributions were thus developed which expressed in terms of $P(E)$, the annual probability of exceeding load level $E$, where:

$$P(E) = A \exp\left(\frac{-E}{E_0}\right)$$

and

$$E = \frac{E_{RP}}{E_{100}}$$

$A$ and $E_0$ are constants which characterise the environment, $E_{RP}$ is the load corresponding to the return period, $RP$, and $E_{100}$ is the most probable 100-year load. These equations are only accurate for the upper tail of the load distribution, i.e. for return periods $> \sim 20$ years. Parameters of a lognormal distribution, fitted using the above approach, have been derived for 20 and 100-year loads appropriate to various geographical areas as follows:

<table>
<thead>
<tr>
<th>Geographical location</th>
<th>Load, E</th>
<th>Mean</th>
<th>Std dev.</th>
<th>COV</th>
</tr>
</thead>
<tbody>
<tr>
<td>20 year return period</td>
<td>CNS &amp; SNS</td>
<td>0.80</td>
<td>0.84</td>
<td>0.18</td>
</tr>
<tr>
<td></td>
<td>NNS</td>
<td>0.75</td>
<td>0.81</td>
<td>0.21</td>
</tr>
<tr>
<td>100 year return period</td>
<td>CNS &amp; SNS</td>
<td>1.00</td>
<td>1.05</td>
<td>0.19</td>
</tr>
<tr>
<td></td>
<td>NNS</td>
<td>1.00</td>
<td>1.07</td>
<td>0.23</td>
</tr>
</tbody>
</table>

Table 8: Parameters of lognormal distribution for 20 and 100 year loads in various geographical areas [Efthymiou et al., 1997]

(where CNS = central North Sea, SNS = southern North Sea, NNS = Northern North Sea)

During an assessment of the failure probability of a jack-up under environmental loading in the central North Sea in 1994, Shell made a comparison of the conventional conservative 100yr. wave + 100yr. current + 100yr. wind approach with their “true” 100-year return loads. It was found that the conservative estimates were a factor of x2.0 on the prediction of base shear and x1.8 on the prediction of OTM [van de Graaf et al., 1994b].
Table 9: Comparison of 100-year and site assessment environmental loads (wind, wave and current) for the jack up unit [van de Graaf et al., 1994b]

6.4.5 Paras Ltd, 1996

Detailed investigations of the estimates of extreme surface elevation derived from analysis of significant wave height and mean water level data recorded at five sites in the North Sea, were reported in 1996 [Spronson, 1996]. Comparisons were made between the estimate of extreme surface elevation derived from crest elevation and mean water level time series data using HSE Guidance notes [HSE Guidance Notes (4th edition), 1990], industry standard and joint probability methods. This study was conducted for HSE and a comparison was made between estimates of

- extreme surface elevation from crest elevation and mean water level time series data using the HSE’s (now obsolete) Guidance Notes, industry standard and joint probability methods.
- return period and the probability of exceedence associated with values of air gap estimate from the extreme surface elevation \( s \) derived using the three methods (air gap as defined in the HSE Guidance Notes is equal to the 50 year surface elevation + 1.5m.)

The approach of interest in this study was the joint probability method. Here, the distribution of extreme values of the combined water level (i.e. mean water level + crest elevation) was evaluated. In the method adopted, the joint density \( f(x,y) \), of variables \( X \) and \( Y \), was estimated from the series of observations of \( X \) and \( Y \) and then the probability of failure, \( P_v \), was estimated using the following expression:

\[
P_v = \text{Pr}[(X,Y) \in A_v] = \int_{A_v} f(x,y) \, dx \, dy
\]

Where: \( A_v \) = the failure region, i.e. the set of \((X,Y)\) such that \( X + Y > v \)

- \( X, x = \) normalised mean water level (m)
- \( Y, y = \) crest elevation (m).

The overall conclusion concerning the joint probability method when compared with the HSE guidance notes method, was that the surface elevations derived from the guidance notes were much greater than those derived from the joint probability technique. This was thought to be due to an apparent over-estimation of the significant wave height when using the guidance notes approach. When the air gap return period was studied, it was found that the
guidance notes approach gave a return period of up to 3800 years, whilst the joint probability approach predicted a return period of up to 5300 years [Spronson, 1996].

6.4.6 Concluding remarks

From the studies reported to date, the ‘traditional’ approach of combining the 100-year wave the 100-year current and the 100-year wind is over-conservative by up to a factor of 2.0 [van de Graaf et al., 1994b, Spronson, 1996], when compared with a joint probabilistic approach.

6.5 Environmental uncertainties and sensitivities

6.5.1 Uncertainties

As discussed above, the environmental parameters have been modelled in different ways by different designers [see also Carr and Birkinshaw, 1989]. In 1985, the main steps in both a deterministic and a probabilistic design process were discussed [Lloyd, 1985]. It was concluded that the introduction of a probabilistic approach and the introduction of environmental uncertainties could mean that the whole design process might have to be adjusted [see also Lloyd, 1990]. It was also noted that “rare forces events, rare storm events and unusual resistance deficiencies” dominated reserve strength requirements [Lloyd, 1985]. The table below summarises the criteria, procedures and practice/reference norms in the deterministic and probabilistic approaches.

<table>
<thead>
<tr>
<th>Approach</th>
<th>Criteria</th>
<th>Procedures</th>
<th>Practice / Reference Norms</th>
</tr>
</thead>
<tbody>
<tr>
<td>Deterministic design process</td>
<td>• Functional requirements</td>
<td>• Morrison equation</td>
<td>• Code allowable</td>
</tr>
<tr>
<td>leading to Implicit reliability</td>
<td>• 100-year wave</td>
<td>• Drag coefficients</td>
<td>• Implicit reserves</td>
</tr>
<tr>
<td></td>
<td>• 100-year wind</td>
<td>• Shielding</td>
<td>• Explicit reserves</td>
</tr>
<tr>
<td></td>
<td>• 100-year current</td>
<td>• Diffraction</td>
<td>(designer “prerogatives”)</td>
</tr>
<tr>
<td></td>
<td>• Foundation conditions</td>
<td>• Linear analysis</td>
<td></td>
</tr>
<tr>
<td></td>
<td>• Soil p-y curves</td>
<td>• System behaviour</td>
<td></td>
</tr>
<tr>
<td>Probabilistic analysis process</td>
<td>• Long term wave, wind &amp; current distributions</td>
<td>• Probabilistic analysis</td>
<td>Past experience</td>
</tr>
<tr>
<td>leading to Deterministic design</td>
<td>• Joint probabilities</td>
<td>• Non linear analysis</td>
<td>Value analysis</td>
</tr>
<tr>
<td>process</td>
<td>• Parameter uncertainties</td>
<td>• System behaviour</td>
<td>Standard society norms</td>
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<td></td>
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<td></td>
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</table>

Table 10: Summary of criteria, procedures and practice/reference norms in deterministic and probabilistic approaches [Lloyd, 1985]

Loading uncertainties in extreme waves were investigated in 1989 [Olufsen and Bea, 1989]. Uncertainties in extreme base shear forces and extreme overturning moments were estimated for an idealised eight-legged jacket structure, which was then exposed to different
environmental conditions for both the Gulf of Mexico and the North Sea. Evaluation of the significance of the wave climate to the response of structures for these locations was carried out. Extreme storm wave heights were modelled using a Gumbel distribution and the ratio between extreme wave height and significant wave height was modelled using a Rayleigh short-term distribution. The wave kinematics were calculated using Airy wave theory. It was found that by treating some uncertainties as dependent from year to year, the COVs of the results were significantly different than if all the uncertainties were assumed independent [Olufsen and Bea, 1989].

Uncertainties associated with extreme return periods of environmental loadings acting on offshore structures were also studied by Bea for the Canadian Standards Authority (CSA) [Bea, 1993b]. The results of this study indicated that, based on the information and data that were available, it was only possible to develop clear characterisations of uncertainties in a small number of cases. It was also noted that the different technical disciplines that were involved in determining environmental conditions and forces recognised and integrated uncertainties into loading characterisations in different ways. Bea also identified a need for a systematic and definitive evaluation of uncertainties in extreme environmental loadings and load effects arising from the need for “design code information sensitivity and consistency in demonstrating compliance with target reliability goals” [Bea, 1993b].

The uncertainty in wave height has been modelled by a number of different distributions by different investigators. These include the Rayleigh [Tromans et al., 1993; van de Graaf et al., 1993; 12], truncated Weibull [Sigurdsson et al., 1994], and the Gumbel [Gierlinski et al., 1993; Olufsen and Bea, 1989] distributions. Uncertainties in the wave period have also been modelled using different distributions, including the lognormal [Olufsen and Bea, 1989] and Rayleigh [Gierlinski et al., 1993] distributions. It has been found that the randomness in the seastate parameters, especially the wave height, is dominant.

Uncertainty associated with the current is not normally found to be significant and this parameter is often treated as deterministic [Sigurdsson et al., 1994; Tromans et al., 1993; van de Graaf et al., 1993; 12]. Moreover, hindcasting of current speed has been performed with reasonable accuracy for speeds in excess of 0.75 m/s with a COV of 7% in the Gulf of Mexico [Gordon, 1991]. However, in a study that utilised the response surface technique, WSAtkins modelled the current speed as an explicit random variable [Gierlinski et al., 1993].
In order to represent the wind loading on a structure, past studies have modelled the wind as either a quasi-static load process or as a deterministic quantity [Gierlinski et al., 1993; 6]. The uncertainty in the wind loading is less significant for structures in shallow waters (contributing to less than 10% of the global loads), but for structures in deeper water, wind loads are more significant [API, 1993a].

6.5.2 Sensitivities

One of the most important studies into the extent of probabilistic modelling required for effective structural reliability assessments was undertaken by DNV in 1994 [Sigurdsson et al., 1994]. This sensitivity study found that where different wave periods were used in the analysis with the same wave height, significant differences in the structural response were noted. It was also noted that the system capacity was not very sensitive to such changes. The significant wave height, $H_s$, was modelled as a truncated distribution, where scale and shape parameters were estimated from storm data collected at a given site. The wave period was found to have a very small COV and was therefore assumed deterministic. A Poisson distribution represented the number of annual storms and a Gaussian distribution represented the sea surface over a short period. The introduction of current was found to increase the basic loading but it was concluded that its presence had little impact on pushover strength [Sigurdsson et al., 1994].

It is generally agreed that the reliability index for structures has been found to be very sensitive to the environmental loading variables. Of these loading variables, the wave height was found to be the most dominant variable, followed by the wave period and Morison’s drag coefficient [Gierlinski et al., 1993; Sigurdsson et al., 1994 and van de Graaf et al., 1994a]. The dominance of these loading variables was found to introduce high correlation between the failure events of different components within a failure path and between failure paths [Gierlinski et al., 1993].

It was concluded that the loading variables together accounted for more than 95% of the total uncertainty [Gierlinski et al., 1993], therefore a rigorous modelling of the uncertainty in these variables is vital for reliability based integrity assessments. This indicated the need for more data collection to develop a joint probability distribution of all relevant environmental parameters. It was shown that the uncertainty in loading modelled through a single random multiplier applied on a deterministic load vector was not adequate for practical applications.
6.6 The ‘airgap’ issue

The so-called airgap is the deck structure clearance above the waves. Thus, the part of the superstructure which was not designed to resist wave impact was required to have a clearance airgap above the design extreme wave crest. In the assessment of the airgap, reference is made to the extreme environmental conditions based on a 50-year return period. The 1990 HSE guidance stated that the airgap relative to the design extreme crest elevation should never be less than 1.5 metres where the air gap was defined as [Smith and Birkinshaw, 1996]:

\[
\text{Airgap distance} = (\text{lower deck height above LAT}) - (\text{maximum wave crest elevation}) - (\text{HAT} + \text{extreme storm surge}).
\]

Where: LAT = lowest astronomical tide, and HAT = highest astronomical tide

It was also recommended that an allowance should be made for the effects of settlement and uncertainties in estimating water depth and extreme crest elevation.

Since the move from a prescriptive to a goal-setting regime, where the guidance notes are no longer mandatory, there was a need to develop rational approaches to airgap determination. This was coupled with a need to determine adequate models for the assessment of wave-in-deck loads on those installations where such loads need to be taken into account. A move towards using a performance standard for airgap, rather than the 1.5 metre assessment method, was suggested by Smith and Birkinshaw, 1996. It was identified that for a particular installation the airgap performance could require that the structure does not sustain damage which could lead to total collapse with a probability of occurrence of greater than 10^-6 per annum, for example [Smith and Birkinshaw, 1996].

Since the withdrawal of the guidance notes, the offshore industry has become more aware of the significance of the air gap issue. A number of studies have been undertaken by HSE in order to improve understanding of the issues surrounding the derivation of the air gap [Smith and Birkinshaw, 1996; BOMEL, 1998b]. HSE are currently planning to set up an industry focus group to discuss the problems and details surrounding the air gap issue.
7. Foundation Modelling

The accuracy of predictions in soil engineering has long been of interest to engineers. As far back as the early 1970s, it was noted that the accuracy of a prediction depended on the quality of the methods as well as on the data used to make the prediction. It was suggested that “in making his prediction, the engineer should be consistent in the sophistication of his method of prediction and in the quality of the data employed” [Lambe, 1973].

Probability of failure is calculated by integration of the probability distributions of load and resistance. It can only provide an absolute measure of reliability when physical uncertainty dominates model prediction uncertainty. In the past, some analyses have shown a significant degree of uncertainty exists about the validity of the foundation model and of the data used for the soil parameters. This uncertainty is sometimes found to be of the same order of magnitude as the physical uncertainty in the environmental load. To derive an absolute value of reliability in such cases, foundation failure was excluded from the analysis based on inspection/observation and sound engineering judgement [Gierlinski et al., 1993; Sigurdsson et al., 1994; Tromans et al., 1993 and Light et al., 1995]. However, this was not always found possible and consequently further investigations into more accurate prediction of the foundation model uncertainty were prompted.

Current axial capacity calculation methods have been derived using data from onshore load tests on small piles [Lacasse and Nadim, 1996; Senner and Cathie, 1993; Pelletier et al., 1993; Foray et al., 1993]. Penetration depth, pile length, pile diameter and ultimate load for the largest piles in such a database are generally smaller than those currently used in the North Sea. The uncertainty is often large because of this. In probabilistic analysis, the “model uncertainty” is defined with a mean and COV and usually a normal or lognormal distribution. Model uncertainty must therefore be evaluated based on comparison pile load tests, deterministic calculations, expert opinions and survey of regulatory organisations, relevant case studies of “prototypes”, results from literature and good engineering judgement [Lacasse and Nadim, 1996; Wu et al., 1989]. The NGI undertook an extensive survey that found that the preferred method of obtaining an estimate of model uncertainty was to evaluate the results of model tests, run specifically to evaluate the expected mechanism of failure [NGI, 1994a].

Hamilton and Murff studied the results of centrifuge tests and performed analyses in order to determine the influence of cyclic lateral loading on the ultimate lateral resistance of
foundation piles in normally consolidated clay. They studied platform foundation reserve strength ratio (RSR) calculations using the ultimate lateral resistance computed using the static criteria described in API RP2A 20th edition. Platforms found to be most favourably affected by this were older shallow water template-type jackets with un-battered piles. With the use of the recommended criteria, the study found that foundation RSR of such platforms was raised by ~30% over that using standard cyclic criteria [Hamilton and Murff, 1995].

More recently, a joint industry funded project on offshore piling was undertaken at Imperial College (IC), London [Jardine and Chow, 1996b; Jardine et al., 1998]. New design approaches were developed for analysing driven piles in clays and sands resulting from a long-term research programme at IC. The new deterministic methods were claimed to be relatively simple and easy to apply in practice and claimed to offer major advantages over the existing API approaches. When tested against a new database of field tests, the formulations “lead to much more reliable predictions for the medium term shaft and base capacities of single piles installed in both sands and clays.” The IC work also drew conclusions concerning time effects and pile group interaction for piles in sand.

7.1 Probabilistic foundation modelling methods

A number of methods have been developed to determine the behaviour of axial piles in foundation modelling. Some of these are based on a probabilistic approach [Tang and Gilbert, 1993; Gilbert and Tang, 1995; van Langen et al., 1995; 58 and Lai et al., 1995] and others use a deterministic approach [Jardine and Chow, 1996b].

A study on the influence of model choice on the calculated reliability of a single pile was performed in 1995 [Lai et al., 1995]. The analyses performed in this study were based on Monte Carlo simulation. It was concluded that there were significant limitations in using the traditional deterministic methods of analysis and that the use of reliability theory could better address the uncertainties associated with variations in soil properties and relative importance of parameters, and enabled the quantification of safety levels.

The key reliability based approaches that have been developed, including Gilbert and Tang’s approach, Shell’s “confidence” approach and Fugro’s probabilistic approach, are considered in the following sections.
7.1.1 Gilbert and Tang Approach

The Gilbert and Tang approach [Tang and Gilbert, 1993; Gilbert and Tang, 1995] uses Bayesian theory to provide a framework for quantifying the model uncertainty given various information levels. The likelihood of a particular model being valid is evaluated by considering judgement and experience, along with the likelihood of observing a set of information if the model is valid. Model uncertainty can be derived using either a first-order or higher-order approach.

This first-order evaluation is a simple approach that focuses on the mean value of a random variable model. The mean value of a random variable $X$, $\mu_x$, is modelled as a random variable $M_x$. The model uncertainty is represented by the distribution of $M_x$. Its standard deviation, $\sigma_{M_x}$, can be approximated by $\sigma_x/\sqrt{n}$ where $\sigma_x$ is the standard deviation of $X$ and $n$ is the number of independent measurements of $X$. Judgement and experience are also valuable in estimating $\sigma_{M_x}$ especially when data is scarce. Random uncertainty is therefore represented as $\sigma_x$ and model uncertainty is represented as $\sigma_{M_x}$. The first-order approach is simple, but it is limited because it neglects uncertainty in other parameters describing the random variable and the probability distribution of the variable.

In the higher-order evaluation method, the theoretical cumulative distribution function is related to the observed cumulative frequency in order to account for uncertainty in the assumed distribution. If a random variable, $X$, is considered, it can be assumed to have a normal distribution and the following linear relationship can be derived:

$$\Phi^{-1}\left(\frac{i}{n+1}\right) = \frac{-\mu_x}{\sigma_x} + \frac{1}{\sigma_x} x_i = c + kx_i$$

Where: $i =$ rank, in increasing order, of observed value $i$

$n =$ total number of test results

$$\left(\frac{i}{n+1}\right) =$ observed cumulative frequency

$\mu_x =$ mean value of $X$

$\sigma_x =$ standard deviation of $X$

$x_i =$ observed value.

Modelling the intercept and slope of this linear relationship as random variables, $C$ and $K$, the following can be obtained:

$$\Phi^{-1}\left(\frac{i}{n+1}\right) = C + Kx_i$$
For convenience, the cumulative distribution function of X, \( F_x(x^*) \), may be expressed as a variable,

\[ \beta = -\Phi^{-1}[F_x(x^*)] \]

Since \( \beta \) is a function of the random variables C and K, \( \beta \) is itself a random variable, B, with the following statistics:

\[ \mu_{B|x^*} = \mu_C - \mu_K(x^*) \]

representing random uncertainty

and

\[ \sigma_{B|x^*}^2 = \sigma^2 + \sigma_C^2 + (x^*)^2 \sigma_K^2 + 2(x^*)\rho_{C,K}\sigma_C\sigma_K \]

representing model uncertainty

Where the statistics C and K are determined using a Bayesian approach and \( \sigma_e \) is the error about the linear model. The error represents the uncertainty in the random variable model for X; thus if X has a normal distribution, then the error will decrease as n increases.

Uncertainties in C and K represent uncertainty in \( \mu_x \) and \( \sigma_x \) due to limited data; they will increase as n increases. Uncertainty in B also depends on \( x^* \); thus as the magnitude of \( x^* \) increases, so does \( \sigma_{B|x^*} \) [Gilbert and Tang, 1995]. The approach allows for either a single random variable or multiple random variables to be included in a reliability analysis.

7.1.2 The “Confidence” Approach

The development of this method by van Langen et al was based on the premise that if the influence of environmental load on pile bearing capacity was excluded, the actual bearing capacity of an offshore pile was an unknown and inherently deterministic quantity [van Langen et al., 1995].

The probability of failure, \( P_f \), was thus defined as:

\[ P_f = 1 - F_L(Q_a) \approx \int_{x=Q_a}^\infty f_L dx \]

Where: \( F_L = \) cumulative probability density function of pile load  
\( Q_a = \) actual pile bearing capacity  
\( f_L = \) probability density function of pile load

The error associated with this definition can be described by an error distribution and, as it expresses the confidence attached to an estimate of the actual pile bearing capacity, it is a ‘confidence distribution’. Confidence bounds for the probability of failure can therefore be established. This approach leaves foundation model prediction uncertainty explicitly visible in the calculated probability of failure.
The pile group capacities adopted in the pushover analyses are obtained from the cumulative confidence distribution of pile group axial bearing capacity. These were derived on the basis of accurate measurement of stresses around a pile during the installation, set-up and load bearing to failure. The shaft capacity of a single pile \( Q_s \) is calculated from the expression:

\[
Q_s = \int_0^L \pi D \tau Dz, dz
\]

Where: \( D \) = diameter of pile

\( \tau Dz \) = ultimate skin friction

The ultimate skin friction, \( \tau Dz \), is defined as follows:

\[
\tau Dz = K \cdot \sigma_v^* \tan \delta
\]

Where: \( K \) = stress ratio

\( \sigma_v^* \) = initial vertical stresses

\( \delta \) = interface friction angle

The stress ratio \( K \) is determined as a function of the over-consolidation ratio, OCR, of the clay and the relative density, Rd, of the sand, while taking into account the distance between the soil element and the tip of the pile, \( Z^* \), and pile radius, \( R \).

Model prediction uncertainties are accounted for by generating a confidence distribution for pile capacity on a statistical model [Lacasse and Nadim, 1996]. Confidence bounds on the distribution of each of the stochastic parameters are given based on the:

- quality and the availability of the data
- degree of interpretation required to determine the parameter
- physical bounds on the value of the parameter.

It is important to note that the confidence distribution of the pile group bearing capacity is derived by assuming that the bearing capacity of the pile group is equal to the sum of the individual pile bearing capacities.

### 7.1.3 Fugro’s probability approach

The probability of a pile failing is a function of pile capacity resistance \( R \), and applied loading \( L \), and was developed [Horsnell and Toolan, 1996]. Thus:

\[
\mu_g = \mu_R - \mu_L \quad \text{and} \quad \sigma_g = (\sigma_R^2 + \sigma_L^2)^{0.5}
\]

Where: \( \mu = \text{mean of normal distribution} \)
The reliability index $\beta$, is defined as the ratio of the mean of the state function divided by its coefficient of variation $V$.

$$\beta = -\ln \left( \frac{\mu_L}{\mu_R} \right) \left\{ \frac{(VR^2+1)(VL^2+1)}{[\ln ((VR^2+1)\cdot(VL^2+1))]^{0.25}} \right\}$$

The probability of failure is then defined as:

$$P_f = \phi (-\beta)$$

Where: $\phi()$ = standard normal distribution function.

Development of this method resulted in the achieved capacity of any particular pile in clay being expressed as:

$$Q = Q_c \cdot f_r \cdot f_d \cdot f_s \cdot f_{ag} \cdot f_i$$

Where: $Q$ = achieved capacity

$Q_c$ = initial design capacity

$f_r$ = factor due to (load) rate effects

$f_d$ = factor due to pile design conservatism

$f_s$ = factor due to sampling effects

$f_{ag}$ = factor due to soil ageing

$f_i$ = factor due to structural interaction

The factor due to load rate effects from dynamic to static capacity is considered appropriate if it is in the region of 1.6. The magnitude of the overall effect of rate effects on pile capacity in clay will be dependent upon the ratio of environmental to gravity load ($W/G$). For a platform in shallow water, this ratio could be 4 or more, giving an upgrade in capacity of approximately 50%.

The factor due to pile design conservatism for a typical Gulf of Mexico soil profile gives rise to a factor of 1.4 when compared with equivalent capacity based upon current API criteria.

The effects of “percussion sampling” on measured values of undrained shear strength in clay, when compared with equivalent push samples, indicates that the strength of push samples can be between 1.3 to 3.3 times higher than driven samples.
The factor due to soil ageing term is used to describe the combined effects of secondary consolidation, thixotropy and sustained gravity loading on the soil surrounding the pile, if these are a function of time and loading condition. For platforms installed over 20 years ago, it is considered that a conservative factor of 1.2 could be applied.

The factor due to structural interaction incorporates the effects of mudmats, which are used to achieve stability of the jacket on the seabed prior to the installation of the piles. If the mudmats remain in contact with the foundation soils during the lifetime of the platform, the combined effect of the mudmat capacity and pile capacity could result in an increase over the nominal design capacity. It is difficult to quantify this effect.

The overall combined effect of all the factors described above leads to a factor between 2.0-3.0 being obtained for piles in normally consolidated clay in the Gulf of Mexico. For North Sea structures designed after 1975, only rate effects and soil ageing would have a significant impact and therefore the combined effect would be a maximum of 1.8.

### 7.2 Deterministic foundation modelling methods

The overall recommendation in API RP2A is that the foundation should be designed to carry static, cyclic and transient loads without excessive deformations or vibrations in the platform. It also suggests that attention is given to the effects of cyclic and transient loading on the strength of the supporting soils as well as on the structural response of piles.

The API recommendations for the design of piled foundations include equations for the ultimate bearing capacity for axial piles as follows:

\[ Q_D = Q_f + Q_p = f \cdot A_s + q \cdot A_p \]

Where:
- \( Q_f \) = skin friction resistance,
- \( Q_p \) = total end bearing
- \( f \) = unit skin friction capacity,
- \( A_s \) = side surface area of pile
- \( q \) = unit end bearing capacity,
- \( A_p \) = gross end area of pile

These recommendations were derived empirically, and for piles in sand were based on the assumption that radial stresses acting on the pile shaft were proportional to effective overburden pressure. For piles in clay, a simple empirical correlation was used which related shear stress with soil parameters such as undrained shear strength or the effective overburden pressure.
The API recommendation for shaft friction and end bearing in cohesionless soils, is to use the following equation:

\[ f = K \cdot p_0' \cdot \tan \delta \]

Where: 
- \( K \) = dimensionless coefficient of lateral earth pressure (ratio of horizontal to vertical normal effective stress)
- \( p_0' \) = effective overburden pressure at the point in question.
- \( \delta \) = friction angle between soil and pile wall

API RP2A LRFD states that it is "usually appropriate to assume \( K = 0.8 \) for both tension and compression loading". However, this has since been considered to be unconservative in some instances, and a so-called North Sea Variant was adopted where \( K = 0.7 \) for compression and \( K = 0.5 \) for tension. [Hobbs, 1993a; Hobbs, 1993b].

Subsequent studies have demonstrated the poor reliability of the API method and skew with relative sand density and pile length. This has lead to unconservative predictions for long piles and loose deposits and conservative predictions for short piles and dense conditions [Jardine and Chow, 1996a]. For piles in clay, the API method was developed from the results of relatively small onshore pile tests that may not be entirely applicable when extrapolated to the very large piles used offshore. Recent research into the effective stress conditions affecting shaft capacity have shown that shaft resistance is sensitive to factors such as pile length, pile material, soil over-consolidation, clay sensitivity, interface angle of friction and direction of pile loading. The approach adopted by API is unable to account for all of these parameters, and independent research has shown that their reliability can be relatively low [Jardine and Chow, 1996b].

The most significant recent work on the development of deterministic predictions of pile capacity has been conducted at Imperial College, IC [Jardine and Chow, 1996a].

### 7.2.1 Imperial College (IC) Method

New methods for the assessment of piles in sand and clay were developed at Imperial College [Jardine and Chow, 1996b]. For piles in sand, the IC method for evaluation of the local pile shaft capacity, was based on the simple Coulomb failure criterion:

\[ \tau_f = \sigma_{reff} \tan \delta_f \]

Where: 
- \( \tau_f \) = peak local shear stress
- \( \sigma_{reff} \) = radial effective stress at point of shaft failure
tan δ.tif = interface angle of friction at failure

Thus the radial effective stress acting on the shaft at failure, \( \sigma'_{rf} \), depends on \( \sigma'_{rc} \), the value acting after installation and full pore pressure and radial stress equalisation, combined with any changes developed during pile loading. The tan δ.tif term represents the critical state sand interface angle of friction, which is developed when the soil at the interface has ceased dilating or contracting. The external shaft capacity is obtained by integrating \( \tau_f \) over the external pile area [Jardine and Chow, 1996b].

A method for the evaluation of the base resistance, \( Q_b \), of closed-ended and open-ended piles in sand was also presented. The base resistance was defined as the total utilisable tip resistance, including the internal skin friction developed by open-ended piles, at a pile head displacement of D/10.

Neighbouring piles can affect the stress regime around a single drive pile. Jardine and Chow [Jardine and Chow, 1996b] used results from tests in which a closed-ended pile was installed at a centre-to-centre distance of nine radii from an individual closed-ended pile. Tests showed that the shaft capacity, Qs, increased by ~50% due to gains in radial effective stresses, while the base response became softer as a result of overall pile uplift. The base resistance associated with peak shaft capacity was noted to fall by ~43%. It was found that, in general, open-ended piles were less strongly affected, but that group effects could also lead to increased shaft capacity and lower base resistance. In the method developed, group interaction imposes a positive effect on axial capacity and it was concluded that this provides an “additional margin of safety through the enhancement of shaft friction.”

In another experiment, large-scale open-ended piles were re-tested six months after piling and then five years after piling, to investigate the effects of time on the capacity. Results showed that the shaft capacity increased by ~85% between the two sets of tests, however, no comparable gains were found for base resistance. The effect of time was represented as follows:

\[
Q_s(t) / Q_s(t=day) = 1 + A \log(t/t=1day)
\]

Where: \( Q_s \) = shaft capacity

\( t = \) time of assessment (up to a maximum of five years)

\( A = \) coefficient (value is 0.5 ± 0.25)
The method proposed was validated by comparison with a new database of 65 high quality pile tests as well as by comparison with predictions from the API RP2A procedure. Reliability assessment comparisons were also carried out with several alternative approaches for predicting the capacity of piles in North Sea dense sands. The results for the predicted results compared to results from the pile tests using the 20th edition API RP2A procedure and the new method are summarised in Table 11.
For piles in clay, the IC method for evaluation of the local pile shaft capacity, was based on
the observation that local shaft failure is governed by the simple Coulomb effective stress
interface sliding law:
\[
\tau_f = \sigma'_{rf} \tan \delta_f
\]
Where:
- \( \tau_f \) = peak local shear stress
- \( \sigma'_{rf} \) = radial effective stress at point of shaft failure
- \( \tan \delta_f \) = interface angle of friction at failure

The radial effective stress at the point of shaft failure, \( \sigma'_{rf} \), is the value of \( \sigma' \), developed at
failure and differs slightly from \( \sigma'_{rc} \) the equilibrium value, by acting prior to loading. Pile
installation and subsequent equalisation lead to \( \sigma'_{rc} \) values that usually exceed “free-field”
horizontal effective stress \( \sigma'_{h0} \) where \( \sigma'_{rc} \) can vary considerably during the potentially
lengthy equalisation period. Incidentally, the existing API recommendations do not take
into account any of the above features, but instead use a total stress approach for calculating
shaft friction [Jardine and Chow, 1996b].

A method for the evaluation of the base resistance \( Q_b \) of closed-ended and open-ended piles
in clay was also derived. The method proposed for a pile in clay was validated by
comparison with a new database of 55 high quality pile tests and by comparison with
predictions from the API RP2A procedure. Reliability assessment comparisons were also
performed with several alternative approaches for predicting the capacity of piles in clay.
The results for the predicted results compared to results from the clay pile tests using the
API RP2A procedure and the new method are summarised in the table below.

<table>
<thead>
<tr>
<th>Method</th>
<th>Mean, ( \mu )</th>
<th>Std dev., ( s )</th>
<th>COV = ( \mu/s )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shaft capacity</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>API RP2A (20th) [API, 1993a]</td>
<td>0.86</td>
<td>0.56</td>
<td>0.65</td>
</tr>
<tr>
<td>IC method [Jardine and Chow, 1996b]</td>
<td>0.97</td>
<td>0.28</td>
<td>0.30</td>
</tr>
<tr>
<td>Base capacity</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>API RP2A (20th) [API, 1993a]</td>
<td>0.77</td>
<td>0.62</td>
<td>0.80</td>
</tr>
<tr>
<td>IC method [Jardine and Chow, 1996b]</td>
<td>1.00</td>
<td>0.20</td>
<td>0.20</td>
</tr>
</tbody>
</table>
Table 12: Assessment of peak clay shaft capacity prediction
[Jardine and Chow, 1996b]

7.3 Foundation uncertainty

A study undertaken in 1995 [Gilbert and Tang, 1995] an approach for evaluating foundation model uncertainty that could then be incorporated into reliability analyses was developed. The approach claimed to provide the ability to estimate statistics of the failure rate, which is clearly important for three reasons: firstly, it represents an objective measure of performance; secondly, it should enter into design decisions since it reflects quality; and thirdly, it can be observed, providing the ability to reduce model uncertainty by analysing observed successes and failures.

There are two main types of foundation uncertainty: random uncertainty and model uncertainty. Random uncertainty leads to the rate of failure for a design, whilst model uncertainty (being the uncertainty in the models used to represent random uncertainty) leads to uncertainty in the failure rate itself. Model uncertainty can be reduced with additional data, and arises due to ignorance about variables and processes that are modelled as random. This “ignorance” may result from having a limited quantity of measurements, a limited quality of measurements or no measurement [Gilbert and Tang, 1995].

7.4 Generic foundation considerations

From the literature studied, it has been shown that piles for which the dead load condition is governing provide a foundation of reduced reliability in sand and normally consolidated (NC) clays, but in over consolidated (OC) clays the reliability meets the API target level. Piles in sand designed to resist environmental loads provide a foundation of reduced reliability, whilst operational experience proves that piles in NC clay in the Gulf of Mexico, designed to resist environmental loads, provide a foundation of acceptable reliability. Dynamic pile testing could improve an understanding of the reliability of some offshore foundations to the API target level. In addition, matching the design to the most appropriate pile tests in the database (and not the whole database) could also improve foundation predictions.

For clay, the NGI recommended that API RP2A 20th edition is used as the preferred method. This method is conservative for NC clay, with a relatively modest COV for NC clay and a slightly higher COV for OC clay. A bias of 1.0 for the OC clay indicates that the method can both under or over predict actual capacity. For sand, the NGI again
recommended the use of API RP2A 20th edition as the preferred method. The method is known to be conservative for dense sand with a high COV. A bias of 1.0 for loose sand indicates that the method can under or over-predict actual capacity [NGI, 1994a].

The IC method developed would appear to perform better than the API RP2A 20th edition method. However, the IC study was undertaken after the NGI review. The apparent performance of the IC method would indicate that this could now be the recommended method. The formulation of expressions to ascertain the pile group interaction for piles in sand is a new concept and, as yet, has not been widely incorporated into foundation analysis. The “confidence approach” described above is a method of estimating the pile bearing capacity with an error associated with this estimation. The Fugro approach estimates the probability of a pile failing in terms of the pile capacity resistance and the applied loading. The Gilbert and Tang method uses a Bayesian approach in order to quantify the foundation model uncertainty. The IC method is derived in a different manner from all these methods, since it uses large databases to establish pile characteristics, which are then developed into formulations for use in reliability analyses.

A potentially large source of modelling uncertainty can be associated with the representation of the foundations. This can be associated both with the foundation model itself but also with the soil parameters used, which can create an incompatibility between the accuracy of the structure and that of the foundation models [van Langen et al., 1995; Mortazavi and Bea, 1996]. Various studies, which include non-linear foundation modelling, have indicated a number of foundation failures. However, these results do not appear to be supported by historical observations where very few platforms are known to have experienced failure due to foundation weakness. This may indicate that the foundation failure predictions are more likely to be a result of the high conservatism in foundation models used [Bond et al., 1997; Jardine and Chow, 1996b]. Some attempts have been made to estimate the bias in foundation analysis using data obtained from the Hurricane Andrew observation [Aggarwal et al., 1996b]. These studies have indicated that the foundation bias factors were significantly higher than the global mean bias factors. A global mean bias of 1.2 was derived, while studies concentrating on different failure modes suggested values up to 1.32 and 0.73 for lateral and axial pile capacity failures respectively. However, these conclusions were only based on a limited number of observations and further work is needed to verify them [Onoufriou and Forbes, 1998].
8. Ultimate Capacity Predictions

The prediction of the ultimate capacity of a system is an essential step in the assessment of structural systems [Nordal, 1990] and hence, reliability. The following section deals with the main issues raised when assessing the ultimate capacity of an offshore platform. The approaches adopted by some of the key investigators are described in the sections below, including those approaches adopted by Shell [Tromans et al., 1993; van de Graaf et al., 1994b and Vanderschuren et al., 1996], DNV/SINTEF [Sigurdsson et al., 1994 and Det Norske Veritas, 1996a] and PMB Engineering [PMB Engineering, 1996].

More recently, the offshore industry has become aware of the different methods that can be used in order to perform the prediction of ultimate capacity. Several ‘benchmarking’ studies have been undertaken recently. These studies have been used to compare the results attained by different organisations that have used different methods and software in order to derive their results. The conclusions of these studies have helped to focus the offshore industry on the need for a more consistent approach and to understand what influences the differences in results [Aggarwal et al., 1996a; Digre et al., 1995; Light et al., 1995; Puskar et al., 1994; Harwood, 1996; Nichols et al., 1996; PMB Engineering, 1993; and PMB Engineering, 1996].

8.1 Different approaches to ultimate capacity prediction

8.1.1 Shell

In the Shell methodology there are several steps to calculate reliability, of which one is to perform a series of non-linear ultimate strength analyses, using USFOS, to establish the load-carrying capacity of the structure and the foundation systems [van de Graaf et al., 1994b; Vanderschuren et al., 1996].

In a Shell study of the Tern platform, it was reported that the design load, $L_d$, was twice the 100-year load ($L_d = 2L_{100}$) and that the ultimate strength, $S_u$, was twice the design load ($S_u = 2L_d = 4L_{100}$). Reference to a plot of extreme load against return period apparently shows that the load will exceed the ultimate strength once in $10^{11}$ to $10^{12}$ years, but taking into account the uncertainty in ultimate strength gives a failure rate of $10^{-10}$ per year [Tromans et al., 1993]. In a study undertaken on the Inde-K platform [Tromans et al., 1993], the design load $L_d = 1.15L_{100}$, and the ultimate strength $S_u = 2.7L_{100}$. Reference to a plot of extreme
load against return period for Inde-K shows that the load will exceed the ultimate strength once in $10^{11}$ to $10^{12}$ years, and taking into account structural resistance uncertainty, the failure rate becomes $10^{-9}$ per year.

### 8.1.2 DNV/SINTEF

In the latest DNV guideline [Det Norske Veritas, 1996a] that is based on results from reliability analyses, the following conclusions were drawn concerning the characteristic features of the ultimate load capacities for offshore structures. Firstly, that the uncertainties in the structural capacity are much less than in the loading. Secondly, that due to the highly correlated load effects, the different failure sequences for the members are highly correlated. Thirdly, that offshore structures are usually not fully balanced and therefore this means that there is one, or very few, failure modes which dominate [Det Norske Veritas, 1996a].

In the DNV/SINTEF methodology, probabilistic uncertainties in structural capacity are assumed to be the yield stresses of the material, the member imperfections (magnitude and direction) and the ultimate capacity, which is determined by Monte Carlo simulation.

If system capacity is directly related to total base shear or total over-turning moment and the load pattern has a minor effect on the system capacity, then system capacity, $SC$, and loading, $L$, can be treated separately [Sigurdsson et al., 1994]. Thus, the annual probability, $P_{f_{sys,annual}}$, that load exceeds the system capacity can be represented by:

$$P_{f_{sys,annual}} = P_{\infty}\{L_{annual} \geq SC\}$$

For a given time, $T_{life}$, probability of total system failure is:

$$P_{f_{sys,total}} = T_{life} \cdot P_{sys, annual}$$

The distribution of the system capacity of the structure, $F_{SC}$, may then be represented thus:

$$F_{SC} = F_{SC}(Z_{structure}, Z_{seastate}, Z_{wave-forces})$$

Where:

- $Z_{structure} =$ a vector of random variables of the structure
- $Z_{seastate} =$ a vector of random variables of the seastate
- $Z_{wave-forces} =$ a vector of random variables of the wave-forces.

The main conclusions that were drawn from a study undertaken by DNV/SINTEF were that the system capacity can be related directly to the base-shear force and can be estimated without taking into account the uncertainty of the load-pattern. It was also concluded that a deterministic description of system capacity is suitable to quantify probability of collapse.
failure and that member imperfection has insignificant influence on the system capacity [Sigurdsson et al., 1994].

8.1.3 PMB Engineering

In the second phase of the PMB Hurricane Andrew study [PMB Engineering, 1996], an improved capacity assessment was developed through case studies and was tested on nine steel jacket platforms. In addition to general improvements in the wave load “recipe”, specific improvements in the analyses of the selected platforms were gained through additional information, such as new hindcast data, site specific soil information, confirmation of platform damage from recent inspections and salvage of platforms.

The type of analysis performed was a static pushover analysis which involved defining a representative profile of lateral forces (wind, wave and current) acting on the platform and then the application of this profile with incrementally increasing amplification factors until the platform’s ultimate capacity was defined. This ultimate capacity was described as the “load level at which the platform is considered to have no additional lateral load carrying capacity”. This ultimate capacity was considered to have been achieved either when a definitive peak in the resistance-deformation curve was obtained, or when the global stiffness of the platform was reduced to a very low value and the displacements at the deck level were in excess of 5 feet [PMB Engineering, 1996].

8.2 Benchmarking studies

A source of uncertainty associated with the substructure strength is the modelling uncertainty. A number of organisations have recently undertaken benchmarking studies, in which different software and/or different organisations were used to predict the ultimate capacity of a number of platforms [Aggarwal et al., 1996a; Digre et al., 1995; Light et al., 1995; Puskar et al., 1994; Harwood, 1996; Nichols et al., 1996; PMB Engineering, 1993; and PMB Engineering, 1996]. The key aspects studied and the main conclusions are outlined in the next section.

8.2.1 Hurricane Andrew JIP

A benchmarking study in the USA was prompted by offshore platform damage caused by Hurricane Andrew in August of 1992 [Aggarwal et al., 1996a; PMB Engineering, 1993; PMB Engineering, 1996]. This incident presented a unique chance “to study the true behaviour of offshore platforms subjected to large hurricanes and to improve procedures used in analytical predictions.” The joint industry funded project had 14 sponsors (13
operators and the US Minerals Management Service MMS), and was carried out by PMB Engineering Inc. Of the 700 installations in the path of Hurricane Andrew, there were 28 jacket type platforms that “suffered substantial damage resulting either in total collapse or rendering the structure unserviceable and beyond repair” and 47 caissons that were “significantly damaged or collapsed”. It was noted that in some instances, platforms were predicted to fail but survived. However, what was unclear was whether this was due to an over-estimation of loads or due to an under-estimation of the strength.

A calibration procedure was therefore adopted which assessed the bias, $B$, in the safety factor for each of three modes of failure (safety factor is resistance to load ratio): jacket bias factors, $B_j$; lateral foundation bias factor, $B_l$, and axial foundation bias factor, $B_a$. The bias factor was thus:

$$\left( \frac{R}{S} \right)_{\text{true}} = B \left( \frac{R}{S} \right)_{\text{computed}}$$

Where: $R =$ resistance (strength) and $S =$ loading.

The computed ratio is that derived from the best practice of procedures and guidelines for ultimate capacity prediction and wave loading analysis. The value of the bias, $B$, greater than unity indicates conservatism in the procedures used. As the information used in obtaining the bias was not definitive, the bias was defined in the form of a probability distribution. The likelihood function, $l_k$, was therefore developed which is the likelihood of $B$ given the outcome, and is defined in terms of the probability of that outcome given the value of $B$:

$$l_k (b \mid \text{the outcome}) = P [\text{the outcome} \mid b]$$

The combined likelihood function of $B$ given the observed behaviour of a number of platforms with a combination of survivals, damages and failures, was defined as:

$$l_k (b \mid n - \text{observations}) = \prod_{\text{platform},i}^n l_k (b \mid \text{observation})$$

Where: $n =$ total number of platforms.

Structural capacity analysis was performed to establish the true response of the platform subjected to the hurricane hindcast conditions. A static pushover analysis was performed using CAP (Capacity Analysis Program) software, in which individual components are allowed to yield and fail and are monitored following failure to assess the impact of their failure on the overall response of the structure.
It was concluded that the foundation bias factors were significantly greater than those for the jacket, indicating significant conservatism in foundation design practices. A study of the bias factors showed that prediction of jacket capacity was moderately conservative. A need to determine failure-mode specific capacity estimates was also identified, which would isolate the impact of uncertainties associated with the modelling of the elements defining the individual mechanisms. This Phase I work also identified areas in the then current platform analysis methods that could be improved [PMB Engineering, 1993]. Phase II was then undertaken to improve the understanding of the biases that were inherent in the state-of-the-practice platform assessment process, in order that improvements to the definition of failure probability of specific platforms, as part of fitness for purpose evaluations are achieved [PMB Engineering, 1996]. For further information on the derivation of safety factors, see [Bertrand and Haak, 1997].

8.2.2 API Task Group JIP

The API Task Group 92-5 JIP was set up to assess the new draft of the API Section 17 guidelines: assessment of existing platforms [Digre et al., 1995]. Eight participants and five engineering firms applied the new guidelines to the same structure. This exercise involved determining the API RP2A 20th edition 100-year load as well as ultimate strength loading [see also references Lloyd, 1988 and Moses and Lloyd, 1993 for development of API RP2A LRFD]. This exercise was undertaken in order to “determine the variability in results for ultimate strength analysis, a key to the Section 17 assessment process.” The companies were asked to select metocean parameters, number of directions for analysis, pile-soil strengths etc. based upon the information in Section 17 as well as the rest of API RP2A. In all, nine different software packages were used for the non-linear ultimate capacity analysis, which were considered to represent the majority of the software used within the offshore industry.

The average variation in the ultimate capacity and reserve strength ratio derived was 23%. The reasons identified for the variations in the results included

- use of static vs. p-y curves to define soil lateral stiffness
- modelling of well conductors to contribute to the foundation capacity
- difference in modelling conductor supports at the mudline

While the COV for ultimate strength (COV=0.16) was within the value assumed by the Task Group in choosing the ultimate strength loading criteria, it was felt that the variations in
capacity estimates and failure modes would “reduce with time as more organisations become familiar with ultimate strength analysis procedures and software”[Digre et al., 1995].

8.2.3 HSE / MaTSU

An HSE study [Nichols et al., 1996] was based on the results from experimental large-scale frame collapse data. A total of 11 organisations participated, performing FE analyses blind with no knowledge of the actual test results. The analyses were based on four tests on two-bay two-dimensional frames. Results and conclusions of the study may not therefore fully cover all issues related to three-dimensional frames. MaTSU studied the FE results from the 11 organisations, where three generic types of modelling behaviour were identified. Results were also assessed in terms of a reserve strength factor (RES = capacity of intact frame / frame design load), and a residual strength factor (REF = capacity of damaged frame / capacity of intact frame). Uncertainties in the results were noted resulting from the use of different software and modelling uncertainties were found to derive directly from the choices and decisions of the individual analyst.

A further study to investigate the possible causes of differences between the organisations took the form of a questionnaire with a follow-up interview with the participating organisations [Nichols, 1996]. Questions concerning the following areas were included: material properties, limitations, resources and reserve and residual strength. Seven different definitions of reserve strength factor emerged and four separate definitions of residual strength factor. It was also noted that some companies did not routinely carry out residual strength ratio assessments “due to the fact that the term does not have a generally accepted definition and clients do not ask to quantify residual strength ratio”.

A paper published in 1997 [Nichols et al., 1997] presented background and results of HSE initiatives to develop understanding of material and geometric non-linear interactions of structural behaviour and also the use of ultimate strength analysis for the assessment of existing structures. It was concluded “ultimate strength analysis is moving from the preserve of research to being an important tool in engineering practice.... Recent developments have demonstrated the potential for ultimate system strength analysis tools to be used reliably as a basis for structure performance standards.” This paper also demonstrated “the limited use generally made of tools to date. It is on that basis that it may be concluded that clear, considered thinking is required up-front for these tools to be an effective aid in evaluating performance.”
8.2.4 **Shell**

Concerned by the apparent difference in results obtained from the use of different software as reported by HSE/MaTSU, Shell undertook its own benchmarking exercise, in which a detailed structural model including all loading and material data was used to provide a common basis, using three different organisations with three different software programs [Harwood, 1996]. The results from this study, based on the Kittiwake structure, showed that the three software packages predicted the same failure path. The ultimate system capacities for all six load cases, differed by a maximum of 13% from the lowest to the highest. All three programs demonstrated significantly different member buckling behaviour, which led to a difference in system capacity being achieved - the difference was found to be mainly due to the modelling of initial imperfections and residual stresses.

8.2.5 **Amoco**

Amoco reported on a similar exercise based on a study of the Lomond platform [Light et al., 1995], in which four different organisations participated. It was found that the derived base shear reserve strength ratios (RSR = lateral load at ultimate strength / design lateral load) ranged from 2.40 to 5.08. Base shear, material modelling and soil modelling were the main three parameters behind the differences in the system strength analysis. Amoco also reported that MMS undertook a benchmarking trial for a Gulf of Mexico platform, in which nine different organisations participated. The RSRs in these trials ranged between 0.74 and 2.47. The differences in results were due to the fact that there was not a common definition of RSR and that wave-in-deck loads were taken into account in only some of the cases.

8.2.6 **BOMEL**

A joint industry funded project, ULTIGUIDE, is currently underway at BOMEL to investigate and explain the potential difference in response predictions for framed structures at component and system levels. Its aim is also to develop scope and format for general structural analysis best practice guidelines [BOMEL, 1997].

8.3 **Generic conclusions regarding ultimate capacity prediction**

The benchmarking studies detailed above have shown that significant modelling uncertainty exists. The studies have highlighted the variations in the assumptions made the competence of the users and human errors associated with the modelling. Some variation is associated with the actual software used, but in most cases, the main source of uncertainty was related to the use of the software rather than its specific characteristics. This type of uncertainty
will reduce as the competence of the user increases. This sensitivity also highlights the need for the development of a framework to provide guidance for consistent application of these methods to reduce the variability in the result [Onoufriou and Forbes, 1998].
9. System Effects

System effects in fixed offshore platforms can be divided into two groups: firstly, deterministic effects which relate to the redundancy of the system, and secondly, effects relating to the randomness of the member capacities. The latter gives rise to a probabilistic contribution to the system capacity. Various studies have shown that under extreme loading conditions, the reliability index of the failure path identified through a deterministic pushover analysis is very close to the value obtained after extensive searches or simulations [Sigurdsson et al., 1994; Kam et al., 1995].

9.1 Deterministic system effects

Deterministic system effects relate to the redundancy built into the structure, which allows load redistribution after the first member failure and results in a higher ultimate load capacity. An important parameter, which can affect the redundancy of the system, is the framing arrangement with X-braced frames being found to offer a much greater redundancy than K-braced frames [Gebara et al., 1998]. Studies to date have shown that the most important system effect contribution comes from the deterministic aspect, i.e. the redundancy of the system, while probabilistic aspects of failure modes and correlation effects make only a small contribution. This is due to the high correlation between failure modes which is observed in fixed offshore platforms. It has been concluded that this indicates that the component based approach, with a deterministic resistance representation (or a COV of the order of 10%) is an appropriate representation within a system reliability assessment [Onoufriou and Forbes, 1998]. Furthermore, this highlights the reserve strength ratio, RSR, as being an important indicator of the system reliability of a structure. Indeed, RSR forms one of the main criteria for re-qualification of offshore platforms [Bea, 1991; Bea, 1993a and API, 1993a].

For a perfectly balanced structure the system effects for overload capacity beyond first member failure are due to the randomness in the member capacities. A balanced structure in this sense refers to a structure where, in a linear analysis, the first member to fail has the same probability of failure as for all other members. For a more realistic structure (i.e. unbalanced), system effects are from both deterministic and probabilistic effects. Deterministic effects are due to the fact that remaining members in the structure can still carry the load after one or several members have failed; probabilistic effects are due to the randomness in the member capacities [Sigurdsson et al., 1994]. The so-called system effect
is, in essence, the difference between the system reliability index and failure of any one member [Gierlinski et al., 1993].

Structural behaviour beyond first member-failure depends on the degree of static indeterminacy, ability of structure to redistribute the load and ductility of individual members. However, structural behaviour is also influenced by aspects such as wave-in-deck loading, the behaviour of the joints [Ma et al., 1995; Sarkani et al., 1995] and the foundation characteristics [Jardine et al., 1998].

In order to assess system effects, there are a number of factors that can be derived from the analysis of a structural model. Three key factors in such studies are the reserve strength, residual strength and redundancy. These are described in detail in the following section.

9.1.1 Reserve strength

The failure of only one part of a system may not limit the capacity of the structure as a whole and a sequence of component failures may occur before the ultimate strength is reached. The reserve strength ratio (RSR) is generally defined as:

\[
\text{RSR} = \frac{\text{ultimate platform resistance}}{\text{design load}}
\]

RSR can be quoted in terms of ratios of platform base shear or overturning moment. For every platform, a different value of RSR will be obtained for every different load case or combination of load cases. It is therefore important to check when assessing RSR values that a full range of load cases has been studied in order to ensure that the most critical case is identified. It should also be noted that in order to make comparisons between different platform’s RSRs that a consistent definition of RSR is used. Different organisations have tended to use slightly different definitions.

An extensive study into the identification, methodology and use of RSR to estimate the overall reliability of offshore platforms was performed by Frieze for HSE in 1993 [Frieze, 1993]. Platform reliability analysis procedures were also reviewed. It was found that not all RSR assessments published in open public literature were comparable, as few had been executed on the same structure, and that RSR and reliability were rarely reported for the same structure. Details of 15 platforms, along with their original design environmental loadings, current reference design loadings, storm loadings experienced, base shear strength and reliabilities were studied.
An extensive study on the definitions and use of the RSR factor was undertaken by Bea [Bea, 1993a; see also Bea and Craig, 1993a]. This study developed a four-tier system for the assessment of structures. The basic definition of RSR used was:

\[
RSR = \frac{R_u}{S_R}
\]

Where: \(R_u\) = ultimate lateral load capacity of platform and \(S_R\) = “reference” lateral loading.

The four tiers in the approach were developed as follows:

- Level 1: “scoring” factor analyses
- Level 2: simplified “limit equilibrium” analyses
- Level 3: modified elastic “state of practice” analyses
- Level 4: “state of the art” non-linear and probability of failure analyses

The primary objective of this system was to allow assessment and re-qualification of platforms with the simplest level 1 method. This was in order to provide a simple, rational and cost-effective approach to the assessment of the RSR of a structure. The more complicated levels in this system would be used for more complex platforms including intense analyses for re-qualification.

Evaluations of fitness-for-purpose, FFP, were based on comparison of the potential “exposures” associated with the platform operations and the RSR based on a given inspection-maintenance-repair, IMR, programme. Thus, if the RSR was “acceptable” then the proposed IMR programme can be implemented and monitored; but if the RSR was “unacceptable” the IMR programme was revised and the FFP re-evaluated until an “acceptable” RSR was achieved. If the RSR was still “unacceptable” then the platform should be decommissioned [Bea, 1993a].

Another definition of RSR as used by Shell [van de Graaf et al., 1993; van de Graaf et al., 1994a] was:

\[
RSR = \frac{\text{environmental load at collapse}}{\text{original design environmental load}}
\]

Shell account for the sources of RSR for North Sea structures from the following contributing factors: explicit code factors, implicit safety in codes, engineering practice, other design requirements and system redundancy and variation in actual material strength.
[Tromans et al., 1993]. The reserve strength resulting from design using working stress
design (WSD) code and conventional design procedures is likely to provide an RSR of ~2.0.

9.1.2 Residual strength

An undamaged structure will have some redistributive capacity, which can be described by
its degree of indeterminacy [Gierlinski and Yarmier, 1992]. The effect of certain damage
scenarios can be assessed by the concept of residual strength. This can be an important
indicator of structural behaviour, and can be defined by the residual resistance factor (RIF)
generally defined as follows [Bolt et al., 1995]:

$$\text{RIF} = \frac{\text{damaged structure’s environmental load at collapse}}{\text{intact structure’s environmental load at collapse}}$$

The ratio of the ultimate capacity of the damaged structure, when compared to the ultimate
capacity of the intact structure, can also give a useful indication of platform behaviour
[WSAtkins, 1997a]. This can be defined as the damage tolerance ratio (DTR), which can be
written as follows:

$$\text{DTR} = \frac{\text{damaged structure’s ultimate capacity}}{\text{intact structure’s ultimate capacity}}$$

Thus the value of the DTR characterises weakening of the structure by the damage. For
example, a DTR of 0.9 would indicate a 10% loss in the reserve capacity.

9.1.3 Redundancy

As previously mentioned, fixed offshore platforms have a large number of load paths such
that failure of a single component does not necessarily lead to full structural collapse. This
observation is accounted for in the “redundancy” of the system. There are various
definitions of the redundancy of a system as follows:

The redundancy factor (RF) was originally defined by Marshall in 1979 as follows [see Bolt
et al., 1995]:

$$\text{RF} = \frac{\text{damaged strength}}{\text{strength loss}} = \frac{N_{LP} - 1}{N_{LP} - (N_{LP} - 1)} = N_{LP} - 1$$

for simple systems with a number of identical parallel load carrying elements ($N_{LP}$).

BOMEL studied the different definitions of redundancy factor and adopted the following
definition for their work on ultimate strength of tubular framed structures [Bolt et al., 1995]:

$$\text{RF} = \frac{\text{ultimate strength}}{\text{ultimate strength}}$$
Redundancy has also been described as the robustness of a structure, where the definition is “the probability of system failure in the presence of damage compared with the intact structure” [Bolt et al., 1995].

According to WSAtkins the redundancy factor is calculated for the intact structure from the load factor for global collapse ($\lambda_{ult}^i$) and for first member failure ($\lambda_{1}^i$). The damage tolerance ratio is calculated from the ultimate load factor for global collapse for the damaged ($\lambda_{ult}^d$) and intact structures ($\lambda_{ult}^i$). Thus the redundancy factor, $RF = (\lambda_{ult}^i - \lambda_{1}^i) / \lambda_{ult}^i$, and damage tolerance ratio, $DTR = \lambda_{ult}^d / \lambda_{ult}^i$ [Gierlinski et al., 1993]. A lower value of the redundancy factor implies that the structure has a high probability of reaching final failure given the initial failure of any one of its primary members.

9.2 Probabilistic system effects

Redundancy can also be expressed in terms of probabilistic redundancy. For example, Cornell [Cornell, 1995] stated that if the ultimate-to-non-linear threshold ratio, $\Omega = 1$, then “in principle an adequately safe structure can be produced.”

9.2.1 Reserve strength

RSR can also be defined based on reliability considerations. Bea used the following probabilistic definition of RSR, based on assuming a lognormal distribution of the load and the capacity:

$$\text{RSR} = \exp(\beta \sigma - K \sigma_{\log})$$

Where: $K = \Phi^{-1}(1-T_S^{-1})$

$\sigma_{\log}$ = Standard deviation of probability distribution of logarithms of annual expected maximum lateral loadings

$T_S$ = return period in years associated with reference environmental lateral loading

$\Phi$ = standard normal distribution

$\sigma$ = resultant uncertainty in the platform capacity and loading ($\sigma^2 = \sigma_{\text{unc}}^2 + \sigma_{\text{disc}}^2$)

$\beta$ = normalised measure of the structural probability of failure

The annual likelihood that the platform capacity is exceeded by the environmental loadings was then defined as, $P_f$, thus [Bea, 1993a]:

$$P_f = 10^{-\beta}.$$
Where: $P_f = \text{probability of failure during one year}$

$\beta = \text{normalised measure of the structural probability of failure}$

### 9.2.2 Redundancy

System effect results can also be expressed in the form of probabilistic redundancy measures, as discussed in [Cornell, 1995]. Furthermore, a complexity factor, a net system factor and a redundancy factor can all be derived in a probabilistic sense, as used by WSAtkins in a detailed integrity assessment study [Gierlinski et al., 1993]. Work in 1994 [Goyet and Saouridis, 1994] also provides for a more in-depth study into the probabilistic redundancy of steel jackets using dedicated software, ARPEJ.

The system effect was defined as the difference between the system reliability index and failure of any one member in a study undertaken by WSAtkins [Gierlinski et al., 1993]. In the case of the intact structure in this study, the system effect was small. The system effects can be represented by different “factors”.

The complexity factor was used to express the effect of a number of elements, relative dominance of elements and correlation between components at Level 1. The closer the value to unity, the higher the correlation between components and hence the relative dominance of only a few members. Thus,

\[
\text{Complexity factor} = \frac{\beta_{L1}}{\beta_{mlfm}}
\]

Where: $\beta_{L1} = \text{reliability index derived for the first failure of any member}$

$\beta_{mlfm} = \text{reliability index for first failure of a member previously identified as critical}$

The net system factor was used to indicate the effect of the overall system. The nearer the value to unity, the smaller the effect of the overall system. Thus,

\[
\text{Net system factor} = \frac{\beta_{mlfm}}{\beta_{sys}}
\]

Where: $\beta_{mlfm} = \text{reliability index for first failure of a member previously identified as critical}$

$\beta_{sys} = \text{reliability index for system failure}$

A probabilistic redundancy factor was also used by WSAtkins, which was defined as follows:

\[
\text{Redundancy factor} = \frac{(\beta_{sys} - \beta_{L1})}{\beta_{sys}}
\]

Where: $\beta_{L1} = \text{reliability index derived for the first failure of any member}$

$\beta_{mlfm} = \text{reliability index for first failure of a member previously identified as critical}$
\[ \beta_{sys} = \text{reliability index for system failure} \]

Such factors are only an indication of relative measures of redundancy and are known to have a number of drawbacks. As such the values derived must be treated with caution. The factors are only applicable to the specific wave approach used [Gierlinski et al., 1993].

### 9.3 Generic system effect considerations

It should be noted that methods used in most of the studies reported to date do not integrate the foundation and joints behaviour in the system assessment. This was identified as an area requiring further work to address these issues and develop a compatible methodology which treats the various failure modes and sources of uncertainty on a consistent basis [Onoufriou and Forbes, 1998]. Other relevant factors, which also need to be re-examined and incorporated in the assessment methods, include the air gap criteria [Smith and Birkinshaw, 1996] and combine extreme environmental and fatigue [Faccioli et al., 1995] or fracture conditions.
10. Case studies

A number of complex and significant case studies have been performed by specialist organisations over the last five years in the structural assessment of offshore installations. Those considered for the purposes of this review have been provided by the sponsors of the project. These reports are considerably more detailed and precise and provide much more depth than literature available in the public domain. This review therefore reports on the analyses and methods for assessment and draws conclusions on the assessment of the following installations:

- Indefatigable 49/18AD [Imm et al., 1989]
- Lomond platform [WSAtkins, 1997a]
- Leman 49/27 AP platform [WSAtkins, 1997b]

In addition, nine different platforms were studied during the Hurricane Andrew Phase II joint industry project [PMB Engineering, 1996] and the key aspects of the study and the results have been included herein.

10.1 Indefatigable 49/18AD

10.1.1 Analyses and methodology

In 1989, Amoco undertook a full structural reliability assessment of the Indefatigable (Inde) 49/18AD platform installed in 1968. A wave height de-manning restriction had been imposed in 1982 due to the high stress levels in the bottom K joints. The reliability analysis undertaken in 1989 included the development of a probabilistic load model, the utilisation of full-scale joint tests and Bayesian updating based on observations over the lifetime of the structure.

At the time of this study, the “ultimate goal of a reliability analysis was to aid engineering decision making” [Imm et al., 1989]. Platform failure was defined as structural failure in the form of “significant platform deflection, resulting from environmental overload.” The probabilistic distribution of the base shear at Inde was determined by the use of an in-house Load Model computer program. The equation that was the basis for the load model is:

\[ L = G \left( \xi H^5 + \epsilon C \right) \]

Where:  
- \( L = \) base shear  
- \( G = \) wave-to-wave force uncertainty, due to variations in drag, shielding etc.
H = wave height
C = current
\(\xi, \delta, \varepsilon\) = constants relating wave height and current to base shear.

The factors L, G, H and C were random variables.

A linear space frame SF, structural analysis computer program was used to assess member behaviour. It was assumed that members behave elasticity until failure, when the member stiffness drops to zero and the member forces becomes a constant, equal to a fraction of their maximum capacity. The fraction therefore may vary from 0 to 1 for compression members, but is 1 for tension members (assuming perfectly elastic-plastic behaviour). Member failure using this program ‘SF1’ was checked only with respect to axial load, with no bending moment interaction included.

A first order reliability method was applied using the computer program SHASYS. After the load profile is input in to ‘SF1’ and the member most likely to fail is identified, the probability of failure of this member is determined using SHASYS. This process is repeated for the next most likely member to fail until platform failure occurs, which was taken as a significant amount of deflection at the top of the jacket. The probability of this failure path is then determined by the program from the individual member failure probabilities.

Site specific oceanographic parameters were developed for use in the pushover analyses [Brown and Root, 1993a]. The wave height was modelled with a Weibull distribution for a 50-year wave of 49.5 feet (15.1 metres). The current was assumed to consist of two components - astronomical tidal current and storm surge and an effective current derived. Wind was accounted for implicitly as a function of wave height. In the broadside direction, the 1 minute wind speed at an elevation of 10m increases from 81 to 115 mph for wave heights from 42 to 62 feet (12.8 to 18.9 metres).

Bayesian updating was undertaken with respect to the probability estimate of the base shear that had occurred over the past 20 years at Inde. A correction distribution, X, was defined which multiplied the original load random variable L as follows:

\[ L' = X \times L \]

Where: L’ = posterior load random variable
L = original random load variable
X = correction value (random variable).
10.1.2 Results and Conclusions

Results of the reliability analysis were presented in the form of failure trees. It was found that the probability of members 1, 3, 5 and 7 failing was 41%, which was found to be the most likely failure path. A combination of all the most likely failure paths exhibited a broadside platform failure probability of 53% over 20 years. A similar failure probability for the end-on direction gave a probability of 79% over 20 years. If the broadside and end-on sectors were independent, the total platform failure probability was calculated as follows:

\[ Pf = 0.79 + 0.53 - (0.79 \times 0.53) = 0.90 \]

This indicated a platform probability of failure of 90% over 20 years.

The Bayesian updating process implied that the originally calculated Inde failure probabilities were too high either due to an overestimation of the load or an underestimation of platform capacity. The platform probability of failure was recalculated to be 38% over 20 years, with an annual probability of failure of 3%, as no joint failure had been observed over the preceding 20 years.

10.2 Lomond

10.2.1 Analyses and methodology

In 1997, WSAtkins undertook a study into the structural system reliability of Lomond [WSAtkins, 1997a] using their software, RASOS. Structural analyses were undertaken including static, linear elastic, component utilisation and non-linear progressive collapse analysis. System reliability analyses were performed for both intact and damage scenarios. The Lomond structural model consisted of three types of load bearing components: leg and tubular members, piles and joints. Secondary components including risers, J-tubes and conductors were modelled as tubular elements and were used only for the environmental load generation. However, the inherent stiffness of these secondary components was not included in any of the response calculations.

Load and resistance models for extreme environmental conditions were described in terms of dead load, seastate parameters, material properties and structural geometry. Structural analyses were carried out - starting with a static linear elastic case, followed by the derivation of component utilisation ratios and then a non-linear progressive collapse analysis. The reliability analysis was performed for both the intact condition and a number of damage scenarios.
The study on the intact structure was performed in four main stages as follows:

- deterministic analysis and non-linear response for 50-yr extreme wave condition
- progressive collapse analysis (using the virtual distortion method) to determine push-over capacity for design and extreme loading conditions
- system reliability using the full failure tree approach for one wave direction
- estimation of system reliability for other wave directions, by representing the resistance variability by a single random variable.

Loading on Lomond was applied as four different types:

- operational loads (in terms of concentrated forces and moments on selected structural nodes)
- environmental loading (combined wind, wave and current) on the jacket, representing the 50-year return conditions
- buoyancy loading on the jacket
- gravity loading on the jacket.

The table below shows an example of the environmental loading applied to the Lomond jacket, and represents the 50-year return conditions.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wave height, H</td>
<td>25.2 m</td>
</tr>
<tr>
<td>Wave period, T</td>
<td>17.37 sec</td>
</tr>
<tr>
<td>Surface current</td>
<td>0.66 m/sec</td>
</tr>
</tbody>
</table>

Table 13: The 50-year extreme storm conditions for Lomond platform (W direction)

10.2.2 Results and Conclusions

In the WSAtkins study, the pushover capacity of the jacket was evaluated under both the design and the extreme environmental loading conditions [WSAtkins, 1997a]. For the latter, wave-in-deck forces were taken into account. Progressive collapse analyses were undertaken for several values of wave height. For each wave height, a crest position was established which corresponded to the maximum value of the total base shear and the associated response of the structure was recorded.

Failure of the intact jacket was defined as the onset of global mechanism formation. The ultimate load factor, equivalent for this loading case to the RSR was determined for both the design loading conditions and the 50-year environmental conditions. Under the extreme
loading conditions, the RSR obtained was lower than that for the design conditions. This was thought to be due to “significant shift of the centre of gravity of the horizontal load towards the top of the structure, and indicates a strong influence of the load pattern on the resistance of the structure” [WSAtkins, 1997a].

Three damage scenarios were studied: Case (i) - brace 5201 removed, Case (ii) - brace 5204 removed, and Case (iii) - both braces 5201 and 5204 removed. The damage tolerance ratio (DTR) was derived for each case. This characterises weakening of the structure (e.g. a DTR of 0.87 indicates a 13% loss in reserve capacity) as follows:

\[
DTR = \frac{\text{Ultimate capacity of damaged structure}}{\text{Ultimate capacity of intact structure}} = \frac{UC_{\text{dam}}}{UC_{\text{int}}}
\]

The damage ratio (DR) was derived for each damage scenario case as follows:

\[
DR = 1.0 - DTR = 1.0 - \frac{\text{Ultimate capacity of damaged structure}}{\text{Ultimate capacity of intact structure}} = 1.0 - \frac{UC_{\text{dam}}}{UC_{\text{int}}}
\]

Results from the Lomond progressive collapse analyses are summarised in the Table 14:

<table>
<thead>
<tr>
<th>Structure</th>
<th>Loading conditions</th>
<th>Results</th>
</tr>
</thead>
<tbody>
<tr>
<td>Intact</td>
<td>Design RSR=3.38</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Extreme RSR=2.92</td>
<td></td>
</tr>
<tr>
<td>Damaged</td>
<td>DTR: Case (i) = 0.86, (ii) = 0.90, (iii) = 0.72</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Extreme DTR: Case (i) = 0.87, (ii) = 0.90, (iii) = 0.74</td>
<td></td>
</tr>
</tbody>
</table>

Table 14: Results for intact and damaged structure for design and extreme environmental loading conditions for Lomond platform (W. direction)

Results from the Lomond reliability analyses are shown Table 15:

<table>
<thead>
<tr>
<th>Structure</th>
<th>Methodology</th>
<th>Probability of failure</th>
<th>Reliability index</th>
<th>Damage ratio, DR</th>
</tr>
</thead>
<tbody>
<tr>
<td>Intact</td>
<td>First failure</td>
<td>4.51e-04</td>
<td>3.15</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>Any first failure (all components)</td>
<td>2.71e-03</td>
<td>2.70</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>Any first failure (exc. pile failures)</td>
<td>9.92e-06</td>
<td>4.10</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>Dominant failure path</td>
<td>1.08e-07</td>
<td>2.25</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>System failure</td>
<td>2.44e-07</td>
<td>5.03</td>
<td>-</td>
</tr>
<tr>
<td>Damaged</td>
<td>Case (i)</td>
<td>1.34e-05</td>
<td>4.20</td>
<td>0.82</td>
</tr>
<tr>
<td></td>
<td>Case (ii)</td>
<td>9.35e-06</td>
<td>4.28</td>
<td>0.67</td>
</tr>
<tr>
<td></td>
<td>Case (iii)</td>
<td>4.43e-05</td>
<td>3.92</td>
<td>1.34</td>
</tr>
</tbody>
</table>

Table 15: Reliability results for intact and damaged Lomond platform (W. direction)
10.3 Leman 49/27 AP

10.3.1 Analyses and methodology

In 1997, WSAtkins completed a study commissioned by Amoco, to perform both deterministic and probabilistic structural analyses of the Leman jacket structure [WSAtkins, 1997b]. Within this study, WSAtkins used their RASOS software with its capability of performing both the non-linear collapse analysis and the system reliability analysis. The main aim of this study was to examine the effect of different damage scenarios on the reserve strength ratio and the system reliability index and to thereby carry out an extensive integrity assessment. The study on the intact structure was performed in four main stages:

- deterministic analysis and non-linear response for 50-yr extreme wave condition
- first member failure and ultimate collapse
- system reliability using the full failure tree approach for one wave direction
- estimation of system reliability for other wave directions by representing the resistance variability by a single random variable.

Several damage scenarios were then investigated in two stages: deterministic collapse analyses and then system reliability analyses. These analyses were used to determine probabilistic damage tolerance ratios.

During the analysis of the Leman 49/27 AP structure, the piled foundation was modelled by a combination of non-linear pile elements and linear support springs in order to represent soil-pile interactions. The structural system of the Leman 49/27 AP jacket-pile has piles running inside the main legs welded to the jacket structure at the top leg joints. In order to model this behaviour, separate beam/column elements were used for piles and legs. Constraints were included at leg node levels to ensure identical displacement of legs and piles in the transverse direction, whilst in the axial direction the legs and piles were allowed to move freely and independently. Loading on the Leman 49/27 AP was applied as five different types:

- self weight of the jacket and appurtenance
- weight of deck equipment
- wind loading on the deck
- buoyancy loading on the jacket
- combined wave and current environmental loading on the jacket, representing the 50-year return conditions.
Table 16 shows an example of the environmental loading applied to the jacket, and represents the 50-year return conditions.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wave height, H</td>
<td>44.9 feet (13.7 m)</td>
</tr>
<tr>
<td>Wave period, T</td>
<td>11.6 sec</td>
</tr>
<tr>
<td>Surface current</td>
<td>1.89 feet/sec (0.58 m/sec)</td>
</tr>
</tbody>
</table>

Table 16: The 50-year extreme storm conditions for Leman 49/27 AP (N direction)

WSAtkins studied all eight different wave directions and both the intact and damaged jacket conditions, in order to examine the effects of reserve strength ratio, system reliability index and damage tolerance. The damage scenarios were selected on the basis that the members either demonstrated failure under the collapse analysis of the intact structure, or that the members were affected by short fatigue lives. These conditions are described in the table:

<table>
<thead>
<tr>
<th>Wave directions</th>
<th>Scenarios studied</th>
<th>Effects examined</th>
</tr>
</thead>
<tbody>
<tr>
<td>SW</td>
<td>Intact jacket</td>
<td>Reserve strength ratio</td>
</tr>
<tr>
<td>S</td>
<td>Damaged jacket (in the critical 270° NW wave direction):</td>
<td>System reliability index</td>
</tr>
<tr>
<td>SE</td>
<td>A - A diagonal brace on frame B</td>
<td>Damage tolerance</td>
</tr>
<tr>
<td>E</td>
<td>B - A diagonal brace on frame C</td>
<td></td>
</tr>
<tr>
<td>NE</td>
<td>C - A horizontal brace on frame B</td>
<td></td>
</tr>
<tr>
<td>N</td>
<td>D - Another diagonal brace on frame C</td>
<td></td>
</tr>
<tr>
<td>NW</td>
<td>E - A combination of scenarios A &amp; B</td>
<td></td>
</tr>
<tr>
<td>W</td>
<td>F - A combination of scenarios A &amp; C</td>
<td></td>
</tr>
</tbody>
</table>

Table 17: Conditions studied for the Leman 49/27 AP platform

10.3.2 Results and Conclusions

The ultimate load factor was determined for each scenario studied. For the intact structure, the ultimate load factor $\lambda_{ult}$ was calculated to be 2.13, which is also equal to the reserve strength ratio (RSR) in this instance, since the reference loading is equal to the design loading. The damage tolerance ratio for each scenario was then evaluated deterministically using the following equation:

$$\text{Damage tolerance ratio} = \frac{\text{maximum base shear of the damaged structure}}{\text{maximum base shear of the intact structure}}$$

The closer the value of the damage tolerance ratio is to unity, the higher the tolerance to damage. The ultimate load factor for the damage scenarios compared with the intact structure and the damage tolerance ratios are shown in the table below. Also included are the annual failure probability and the reliability index for each scenario - both these factors were derived from the system reliability analysis using the calibrated response surface developed for the Amoco load model.
Table 18: Results obtained for the intact and damaged Leman 49/27 AP platform

The main conclusions in this study were that the Leman 49/27 AP in its intact condition exhibits satisfactory resistance and reserve strength against the design load and that, whilst there is no established minimum acceptable level of failure probability, the intact structure would appear to have “satisfactory” system reliability. For the damaged structure, the worst case damage tolerance ratio was 0.75 for a single member failure (case D) and 0.66 for a double member failure (case E), both of which were considered to be exhibiting sufficient ultimate capacity in the damaged state.

10.4 Hurricane Andrew JIP

10.4.1 Analyses and methodology

In 1996, Phase II of the Hurricane Andrew Effects on Offshore Platforms JIP was completed [PMB Engineering, 1996]. Nine platforms were selected from the population of “heavily loaded” structures in the Gulf of Mexico in the direct path of the hurricane. Three platforms were selected from each of the three categories adopted in the study. The survival category was derived for those platforms that showed capacity exceeded load albeit by an unknown amount and that no damage or only minor non-structural damage was identified. The ‘damaged’ category was for those cases where known damage to that jacket was identified and the foundation was assumed to be intact, or where damage was known but not specifically identified or attributed to the jacket or foundation. The third category of failure was where known failure of the jacket was identified and the foundation assumed to be intact, or where failure was known but not specifically attributed to the jacket or foundation.

In this study, PMB used an ‘Andrew’ wave height of 18.55m (60.86 ft) to generate the pushover load pattern and wave-in-deck forces. The incremental loads were determined for wave heights from 15.54m to 19.20m (51 ft to 63 ft) at increments of 0.61m (2 ft). Loads were also determined for three additional wave heights of 9.14m, 12.19m and 15.09m (30 ft, 40 ft and 49.5 ft) which were chosen to complete the wave height vs. load curves. A comparison of pushover load profiles was derived for each of the nine platforms studied.
A calibration exercise was then carried out. The objective of this calibration was to determine a bias factor that could be used to improve the analytical process to more closely match true platform behaviour under extreme storm conditions. The calibration process involved a comparison of platform performance determined analytically with that observed following a severe storm or hurricane. All nine platforms were used in this calibration exercise in order to determine multiple bias factors, applicable to both the jacket structure and its foundation, using mode specific capacity predictions.
The probability of survival \( (P_s) \) with no damage was computed for the following condition:

\[
P_s = P \left[ \text{Andrew load level during hour-1 and hour-2} < \text{Capacity level associated with the first predicted event in the jacket and its foundation system} \right].\]

The probability of failure \( (P_f) \) was formulated as follows:

\[
P_f = P \left[ \text{jacket collapsed | foundation survived} \right] \times P \left[ \text{foundation survived} \right]
\]

Or

\[
P_f = P \left[ \text{Andrew load level in hour-1 or hour-2} > \text{Ultimate capacity of jacket or its foundation system} \right].\]

The damage calibration conditions were formulated as follows:

\[
P_f = P \left[ \text{Andrew load level in hour-1 or hour-2} > \text{Capacity level at jacket damage or damage to multiple piles} \right].\]

### 10.4.2 Results and Conclusions

An improved understanding of capacity analysis was developed through examination of the nine case studies. In addition to general improvements in the methodology, specific improvements in the analyses of the selected platforms (compared to Phase I) were gained through additional information - in particular new hindcast data, site-specific soil data and confirmation of platform damage from new inspections and salvage of platforms. It was concluded that these improvements reduced the uncertainties in the predictions of platform behaviour during Hurricane Andrew. The resulting structural analyses were found to match very closely with the post Andrew inspections. These improved predictions of the behaviour of the platforms during Hurricane Andrew were concluded to be “due to the following factors [PMB Engineering, 1996]:

- General reduction in the Andrew load level estimates using the new hindcast
- Explicit joint strength and stiffness modelling
- Realisation of significant differences in the biases in the strength characterisation for the pile/soil and jacket elements.”

### 10.5 Summary of Case Studies

A summary table of the case studies reported in this section highlighting the main effects examined is given in Table 19:

<table>
<thead>
<tr>
<th>Structure &amp; Reference</th>
<th>Effects examined</th>
</tr>
</thead>
<tbody>
<tr>
<td>Indefatigable 49/18AD</td>
<td>Reliability index. Bayesian updating</td>
</tr>
<tr>
<td>[Im et al., 1989]</td>
<td></td>
</tr>
<tr>
<td>Lomond</td>
<td>Reserve strength ratio. Reliability index. Damage tolerance</td>
</tr>
<tr>
<td>[WSAtkins, 1997a]</td>
<td></td>
</tr>
<tr>
<td>Leman 49/27 AP</td>
<td>Reserve strength ratio. Reliability index. Damage tolerance</td>
</tr>
<tr>
<td>[WSAtkins, 1997b]</td>
<td></td>
</tr>
<tr>
<td>Hurricane Andrew JIP</td>
<td>Damage tolerance. User influence. Use of hindcast data</td>
</tr>
<tr>
<td>[PMB Engineering, 1996]</td>
<td></td>
</tr>
</tbody>
</table>
Table 19: Summary of case studies and the effects examined
11. Offshore Reliability Approaches

During the structural design of offshore platforms, reliability assessments can be undertaken in order to account for fluctuations in loads, variations in material properties and uncertainty in the structural models used. The probability that the structure will not perform as intended is in fact the probability of failure for a certain load situation. Reliability of the structure can be defined as the compliment of this probability of failure and can be used as a measure of safety, or as a useful decision variable. In a 1995 study conducted by AME, it was concluded that “the reliability of the bracing components is only one contributory part of the overall system reliability, and the reliability of the structural system may also be significantly influenced by the tubular connections, as a result of ultimate strength and fatigue, the foundation and the airgap” [AME Ltd, 1995].

The probability of failure, \( P_f \), is calculated by integration of the probability distributions of load and resistance and can be defined in general terms as follows:

\[
P_f = \Phi(-\beta)
\]

where: \( \Phi() \) = standard normal distribution function

\( \beta \) = reliability index

\( P_f \) and \( \beta \) can be calculated by a reliability method which can be any of several available methods, including approximate analytical methods, such as first and second order reliability methods [see Gierlinski et al., 1993], as well as simulation methods, such as the Monte Carlo technique. Monte Carlo simulation is a method for obtaining information about system performance from component data which has also been referred to as synthetic sampling or empirical sampling. It consists of building many systems by computer calculations and evaluating the performance of such synthesised systems. Reliability analysis for offshore structures involves the generation of directional long term statistics of extreme load, the calculation of the ultimate strength of the structure for various directions, an estimation of uncertainty in the structural strength and then finally the calculation of the probability of failure.

11.1 Methods

A structural system with multiple failure paths can be represented by a series of parallel sub-systems where each sub-system represents a failure mode. The combined structural reliability of the system can be calculated from the constituent component reliability, using methods such as those given by [Thoft-Christensen and Murotsu, 1986]. In the case of
complex structures such as offshore platforms, where there is a very large number of possible failure paths, there may be a large number of participating components which makes this approach impractical. For this reason, research efforts have been focused on the development of more efficient system reliability methods for these structures. Some of the main methods proposed for fixed offshore platforms are discussed in the following sections.

11.2 Search algorithms based on probability criteria

Rigorous system reliability analysis requires substantial computational undertaking and ways to develop efficient methods for identifying the most dominant failure paths and deriving the combined system probability of failure have been advanced. The objective is to develop efficient methods for identifying the most dominant failure paths and calculating the combined system probability of failure. Such methods include the selective enumeration technique [Shetty, 1994], the branch and bound method [Karamchandani and Cornell, 1987], the marginal probability and leading probability methods [Thoft-Christensen and Murotsu, 1986]. These methods search for the most dominant failure paths according to probabilistic criteria. A review and detailed discussion of such methods were performed in 1993 [Kam et al., 1993].

WSAtkins used system reliability analyses to identify dominant failure modes and to calculate system reliability measures [Gierlinski et al., 1993]. The method applied is basically stochastic modelling, with the reliability analysis being based on the first order reliability method (FORM) approach. Random variable probability models are used for describing the uncertainty in basic variables. All of the important environmental parameters are modelled as explicit random variables. The uncertainty models used for tensile and compressive yield stresses are the same as in Nordal et al., 1988, who used a simplified FORM approach.

WSAtkins use their analysis package, RASOS, which utilises a joint beta-point concept for reliability formulation combined with a virtual distortion method (VDM) technique for non-linear structural analysis. The VDM concept uses the “superposition principle where any given structural condition is derived from a combination of two states - a fundamental state from the original linear elastic solution and a virtual state caused by virtual distortions introduced into the structure to account for the non-linearities” [Gierlinski et al., 1993]. The joint beta-point for a failure sequence is determined as a solution of a multi-constrained non-
linear optimisation method. This enables the use of more realistic member post-limit behaviour models and combinations including more than one failure mode per structural element.

Failure-tree enumeration is carried out to obtain close bounds on system reliability. The lower bound on system reliability is the reliability index for first failure of any member and the upper bound is found by analysis of all the dominant load paths identified [Shetty, 1994, WSAAtkins, 1997a].

11.3 Pushover analysis assisted by simulation, sequence or response surface methods

One method of deriving the most dominant failure path is performing pushover analyses. The most critical elements are identified in the analysis, but no account is taken of the effect of possible variations in component strength, which may result from different sequences of failure, and by different combinations of elements. Simulation methods were used in combination with pushover analysis to address these possible variations and their effect [Sigurdsson et al., 1994]. The difficulty associated with this approach is the limited number of simulations which can be performed, given the size of the problem and the high computational demands. Although this is not necessarily a practical approach for reliability assessment of fixed offshore platforms, some studies have been undertaken using this method which have produced useful conclusions and guidelines for simpler approaches.

Simulation methods were used in combination with pushover analysis by DNV/SINTEF in 1994 [Sigurdsson et al., 1994]. The DNV/SINTEF approach used the program USFOS for non-linear structural collapse analysis, and PROBAN for probabilistic analysis. PROBAN was used to perform the reliability calculation and to generate outcome of the stochastic parameters in the simulation studies of the ultimate capacity of a structure [Sigurdsson et al., 1994]. The study concluded however, that the resistance could be treated as deterministic.

The annual system failure probability was determined as the annual probability that the load would exceed the system capacity, thus:

$$ P_{f_{sys,annual}} = \int_0^\infty F_{SC}(x).f_{L_{annual}}(x).dx $$

Where: $F_{SC}(.) = \text{cumulative annual probability distribution of the system capacity}$

$f_{L_{annual}}(.) = \text{probability density function of the annual probability distribution of load}$
Investigations performed by DNV/SINTEF found that the system capacity can be related directly to the base-shear force, and can be estimated without taking into account the uncertainty of the load-pattern. It was also concluded that reliability is dominated by uncertainty in \( Z_{\text{seastate}} \), (a vector of random variables modelling the uncertainties in the seastate description) especially the significant wave height.

A refinement in the deterministic pushover approach was applied in a study undertaken by Shell where the sequence effects within a failure mode were studied [Tromans and Van de Graff, 1992]. Various sequence combinations of the elements participating in the failure mode were studied, as identified by a pushover analysis. A large number of analyses were required to cover the various combinations and in this case a method based on linear superposition of constraint loads was adopted to perform the pushover analyses instead of a non-linear analysis program [Tromans and Van de Graff, 1992; Stewart and Van de Graff, 1990].

A response surface technique (RST) or response surface method (RSM) can be defined as an approximation of the mechanical behaviour of a system by simple functions, where these functions are obtained from sensitivity analyses of the system, thus providing sufficient information on the system behaviour. Once this response surface is suitably defined, any advanced probabilistic method for reliability analysis can then be applied. It should be noted that the response surface does not represent the physical model exactly, but if the approximation of the physical model is selected carefully, the results of the final reliability analysis are found to be close to the results obtained by an exact method, such as a Monte Carlo numerical simulation [Enevoldsen et al., 1994, Chryssanthopoulos, 1992.]

This approach was used in a study by MSL Engineering in which the aim was to compare the reliability of fixed offshore structures to the reliability of jack-ups, the reliability technique used was the response-surface method. This method generated a failure surface by systematically varying each of the important basic variables in turn about their mean values and determining the ultimate strength in each case via a pushover analysis or similar. By fitting an equation to this surface, a strength model is created. It is a function of the basic resistance variables and so can be readily input into a reliability analysis. “The choice of basic variables and modelling accuracies used to create a response surface will be influenced by whether their mean values and/or uncertainties (COV) affect the reliability
outcome. Where the variable can be treated as deterministic, it need not appear as a variable in the response surface. Its deterministic value, however, may be required in the generation of the surface” [Frieze et al., 1999; MSL Engineering, 1995; MSL Engineering, 1997].

11.4 Simplified system reliability methods

Bea developed several approaches for evaluating the acceptable, tolerable or desirable reliability of a structure [Bea, 1991; Bea, 1993a and Mortazavi and Bea, 1996]. The most recent approach developed by Bea, reported in 1997, was applied to the reassessment and re-qualification of two Gulf of Mexico platforms [Bea et al., 1997]. The analysis procedure consisted of three levels of analysis as developed in the API guidelines for reassessment and re-qualification of steel template-type offshore platforms:

i) Screening analysis,

ii) Design level analysis (DLA),

iii) Ultimate strength analysis (USA).

The three levels of analysis were performed sequentially, with the checks becoming more detailed and less conservative. Bea et al reported on two types of analysis - the DLA using the program StruCAD*3D and then the USA using the program ULSEA (Ultimate Limit State Equilibrium Analysis) developed in 1995 by Bea.

- **DLA** - Structure loading and capacity were calculated using the API RP2A WSD (1993); soil structure interaction was evaluated from pile geometry and soil characteristics [see also Leira et al., 1994]. The DLA capacity of a member was determined by the creation of the first plastic hinge or member yielding.

- **USA** - The platform’s lateral loading capacity was determined by using plastic hinge theory. The structure was divided into three primary components: deck legs, jacket and pile foundation. A horizontal shear capacity was formed for the platform once the ultimate lateral capacity had been determined for all three primary components, which was then compared to the storm shear profile. The static ultimate lateral capacity corresponded to when the storm shear just exceeded the platform’s horizontal capacity.

A reliability study was performed to evaluate the implications of the uncertainties associated with the loadings and the various failure modes in two platforms. To compute the probability of failure of the platforms, each of the conditional probabilities (conditional on wave height) of failure were multiplied by the probability of occurrence of seastates that
would generate expected maximum wave heights (equal to the long term distribution of the expected maximum wave heights for the location) and then summed.

A simplified system reliability method was developed by Bea, based on a series system where the components in series were the deck, each platform bay and the foundations [Bea et al., 1997]. Within each component there were parallel elements including deck legs, braces, joints and piles. In order for the component to reach failure, all the parallel elements must have failed. The method was provisionally intended to be a simplified method to be used as a screening tool or as a design optimisation method.

Further work was undertaken to refine this method further, and also to compare it with some of the more rigorous approaches used. Results from the simplified analyses were compared to results from 3D linear and non-linear analyses. It was found that the simplified procedure predictions on loadings and capacities compared well to the more elaborate methods [Bea et al., 1997].

In 1994, Cornell also worked on the development of a “random-variable level probabilistic model of structural demand, behaviour, and capacity”. This work was based on “near-failure, static/dynamic, displacement behaviour of structural systems and exploits an explicit analytical form” [see also Cornell, 1994]. One of the main conclusions of Cornell's review was that “it is desirable to set a quantitative structural reliability level or levels as the objective and starting point for any structural criteria” since “many benefits of clarity, consistency and efficiency can follow from that beginning” [Cornell, 1995].

In a study by AME, [Walker, 1997], a new method for assessing the strategic level of structural safety was introduced. A need was identified to have a form of modelling which could accommodate the aspects of structural safety with both technical and human factors, in combination with the mechanical aspects of reliability analysis. This new model “modifies and augments the detailed approach to structural safety evaluation using reliability analysis. The central concept of the model is called ‘structural toughness.’ The toughness aspect reliability analysis is intended to augment the current reliability analysis by assessing if the structure will indeed be safe under conditions which vary from the idealised conditions incorporated in the reliability calculations approaches.”
This structural toughness, $\tau$, was therefore defined as: “a measure of a structure’s ability to sustain perturbations in loading, geometry and material properties within the design system without loss of intrinsic safety.” Structural toughness would be generally evaluated by identification and review of relevant research, review of design/fabrication practice, review in change in use, and finally, in review of developments in education and training. More specifically, $\tau$ would be evaluated by performing perturbation analyses on the limit state of the structure, along with performing reliability analyses for the generic structure. It should be noted that this study presented the model in its formative stage, and significant development is envisaged before the structural toughness model concept can be transformed in to a working approach.

### 11.5 Discussion

The above section has shown that different individuals and organisations have developed many distinct methodologies for the derivation of reliability for offshore structures. The approaches developed by Shell [van de Graaf et al., 1994b], DNV/SINTEF [Sigurdsson et al., 1994] and Cornell [Cornell, 1995] all focus on the same central probability theory. This is that the probability of survival of the structure is based on the probability that the environmental loading in a particular wave direction does not exceed the collapse load of the structure. Table 20 below summarises the probability equations used in these methods.

<table>
<thead>
<tr>
<th>Method</th>
<th>Probability equation</th>
<th>Definitions</th>
</tr>
</thead>
</table>
| Shell                   | $P(\text{survival}) = P_0 (L_0 < \lambda_0 \cdot S_{\text{ref}})$                    | $L_0$ = environmental load in direction $\theta$,  
$\lambda_0$ = collapse load factor in direction $\theta$,  
$S_{\text{ref}}$ = reference base shear force,  
$\theta$ = wave attack direction |
| DNV/SINTEF              | $P_f \approx \int_{0}^{\infty} F_{SC}(x) \cdot f_{L_{\text{annual}}}(x) \cdot dx$    | $F_{SC}(\cdot) = \text{cumulative annual probability distribution of the system capacity}, f_{L_{\text{annual}}}(\cdot) = \text{probability density function of annual probability distribution of the load}$ |
| Cornell                 | $P_f = \int_{H(R)}^{\infty} e^{-S} f_R(S)dx = H(R)$                                | $P_f$ = probability of failure,  
$H$ = complementary cumulative distribution function (CCDF) of load,  
$f_R$ = probability density function of the capacity or resistance,  
$\delta_R$ = coefficient of variation of capacity,  
$R$ = mean capacity |

Table 20: Summary of probability equations used
 WSAtkins used their program RASOS to develop a failure-tree enumeration to obtain close bounds on system reliability. The MSL approach used a response surface method to generate a failure surface, followed by the development of a strength model. The AME approach has been developed based upon a new concept of “robustness” - this requires further effort to develop it into a quantified method.
12. Discussion on review study

The aim of this report was to describe the status of system reliability assessment. The key underlying question throughout this study was to identify what changes or improvements could be made to the reliability assessment process in order to move towards more consistent reliabilities. The report covers an introduction to the problems associated with reliability assessment, briefly describes generic reliability issues and, in particular, the reasons behind uncertainty and sensitivity. Sections then introduce all the major aspects of reliability analysis and a number of case studies are then reported, along with an investigation into the different reliability approaches currently used offshore.

The review study presented here was an attempt to gain a historical appreciation and understanding of the current techniques and the philosophy behind them, as applied to the performance of structural reliability analyses of offshore structures. The thrust of this study is the need to move towards approaches that are more consistent reliability and an increased understanding and decreasing level of uncertainty. The key issues that were identified in the review study are highlighted here. They have been segregated into those that are generic and those that are applicable to the specific example of fixed offshore structures and are presented in tabular form in the following sections.

12.1 Summary of generic issues relating to offshore structures

The main qualitative aspects identified that relate to offshore structures are briefly summarised in the following sections:

Probability of failure

- Reliability involves dealing with events whose occurrence at any time cannot be predicted. The probability of occurrence is expressed by likelihood of the event occurring:
  \[
  P(f) = \Phi(-\beta)
  \]
  Where:  \( \Phi() \) = standard normal distribution function
  \( \beta \) = reliability index.
- Probability of failure is the integration of probability distributions of load/ resistance. Absolute measure of reliability is only obtained when physical uncertainty dominates over model prediction uncertainty.
- Reliability analysis for offshore structures involves the generation of directional long-
term statistics of extreme load, calculation of ultimate strength for various wave
directions, an estimation of uncertainty in the structural strength and then calculation
of the probability of failure.

Uncertainties and sensitivity

- Uncertainty is categorised into three main groups: physical, statistical and modelling
  uncertainty. Physical uncertainty arises from the actual variability of physical
  quantities, such as loads. Statistical uncertainty arises due to a lack of information.
  Model uncertainty occurs from simplifying assumptions not included in the structural
  analysis model.
- There is also a degree of user uncertainty – this becomes more critical when the
  activity being undertaken has high uncertainty in methodology or is highly sensitive to
  the overall reliability result.
- Sensitivity gives an indication of significance of a factor in affecting overall
  reliability. Investigating relative sensitivity involves a study of the effect of each
  different parameter on the results.

Better quantification and reduction of uncertainties

- When reliability of a structure is determined, it is the most accurate prediction for a
  specific structure, foundation, location, environmental conditions and software used.
- Changes in modelling/software have helped minimised errors. However, modelling
  uncertainty still needs to be addressed. Progress in predicting environmental
  conditions has led to improved precision in the representation of environmental loads.

Improving consistency in assessments

- Comparison of structural reliability must be approached with caution as the data,
  methods and assumptions used have changed in the recent past and vary from user to
  user - any comparisons undertaken must be strictly on a like-for-like basis.
- To improve consistency of results between different structures/users, increased
  awareness of uncertainties/sensitivities at each step of a reliability analysis is needed.
- Development of a framework to identify main steps, with justifications, will go
  towards improving overall structural reliability and consistency.

Competence and guidance for users

- User uncertainty is affected by competence, which is more critical when the activity
  has high uncertainty or is highly sensitive.
- Need to move towards guidelines for a more rational approach. The use of different
  models/ software/ users and variations in methods/assumptions lead to different
uncertainties being included in analysis.

- Need to reduce/better quantify modelling uncertainty, and consider improved means of incorporating it into reliability analysis.

Interpretation of system effects

A number of different factors can be studied in order to assess system effects derived from the analysis of detailed structural models. Factors include reserve strength, residual strength and redundancy.

Need for framework

A number of studies on idealised behaviour of structures here identified the need for some kind of framework or general procedure in order to assess offshore platforms with a range of brittle and ductile behaviours, and a variety of failure modes, but with a more rational and consistent approach e.g. [Onoufrion et al., 1994; Birkinshaw et al., 1994]

12.2 Summary of issues specific to fixed platform type offshore platforms

The main qualitative and quantitative aspects identified that relate specifically to fixed platform type offshore structures are briefly summarised in the following sections:

Treatment of drag/inertia and marine growth

System capacity can be estimated without taking into account randomness in inertia and marine growth coefficients. The inertia and marine growth coefficients can be modelled as deterministic. Uncertainty in the drag coefficient cannot be ignored. The COV for the drag coefficient can be in the order of ~20% [Gierlinski et al., 1993].

Loading uncertainties

- Loading variables can account for up to ~95% of total uncertainty (when foundations are ignored) and a rigorous assessment of these variables is vital for reliability based integrity assessments. Need more data to develop joint probability distribution of all environmental parameters.

- Uncertainty in loading modelled through a single random multiplier applied on a deterministic load vector is not adequate for practical applications. The loading can be represented with a COV in the order of 15% [Gierlinski et al., 1993].

Resistance uncertainties
• Despite the fact that response variables generally are less dominant than loading variables this is only true for cases where foundations have been ignored. For those cases where foundations are included in the analysis the response uncertainties require assessment and can be of the same order of uncertainty as the loading.

• Analyses have shown a significant degree of uncertainty exists about the validity of foundation modelling and of data used for soil parameters. This uncertainty has been sometimes found to be of the same order of magnitude as the physical uncertainty in environmental load [Birkinshaw and Smith, 1996].

• The foundation capacity derived by the API method has a COV ~32%, and by the IC method has a COV ~22% [Jardine and Chow, 1996b].

• Yield strength is also a key factor for the resistance, and can be represented with a COV in the order of 5% to 12% [Gierlinski et al., 1993; Sigurdsson et al., 1994].

Modelling uncertainties

• Modelling uncertainties can arise due to the uncertainty from imperfections and idealisations made in physical model formulations for load and resistance, and from choices of probability distribution types for the representation of uncertainty [DNV, 1992].

• It can be described as random factors in a physical model used for representation of load or resistance quantities and can be derived by the ratio between the true quantity and the quantity as predicted by the model. A mean value not equal to 1.0 expresses a bias in the model. The standard deviation expresses the variability of the predictions by the model.

• For example, the modelling uncertainty in piled foundations in sand assessed using the API method has been shown to exhibit a bias of 0.84 and a standard deviation of 0.56. However, if the IC method is used, the bias is 0.97 and the standard deviation is reduced to 0.28 [Jardine and Chow, 1996b].

Environmental extremes

Conventional treatment of waves, current and wind forces was to take each factor separately and then combine the independent extremes simultaneously. This is over-conservative and results in an over-estimation of the design loads required. This over-estimation on base shear has been found to vary between 4% to 25% [Prior-Jones and Beiboer, 1990]. Recently, the development of more reliable
databases of hindcast environmental data has enabled a joint description of these quantities to be determined.

Wave approaches
Generally, only 1 or 2 wave approaches are used in structural platform analysis. For a full analysis, more wave directions need to be assessed. It has been shown that certain more extreme wave approaches, combined with certain more susceptible structural configurations can lead to an overestimation of the ultimate base shear in the order of a factor of ~2 [van de Graaf et al., 1994b].

System effects
- Structural behaviour beyond first member-failure depends on degree of static indeterminacy, ability of a structure to redistribute load and the ductility of members. For a perfectly balanced structure the system effects for overload capacity beyond first member failure, are due to the randomness in the member capacities. For structures that are more realistic, system effects are both deterministic and probabilistic.
- Deterministic effects are from remaining members in the structure which still carry load after 1 or more members have failed; probabilistic effects are from the randomness in member capacities [van de Graaf et al., 1994a]. The system effect is the difference between the system reliability index and failure of any one member [van de Graaf et al., 1993]. The reserve strength ratio will be expected to be in the order of ~2 [Tromans et al., 1993].

Airgap
Need to improve understanding of the issues surrounding the derivation of the airgap [Smith and Birkinshaw, 1996; BOMEL, 1998b]. HSE are currently planning to set up an industry focus group to discuss the problems and details surrounding the airgap issue. In the past, the airgap had to be greater than 1.5m [HSE Guidance Notes (4th edition), 1990]. More recently, it has been defined with a probability of occurrence of less than say ~10^{-6} [Smith and Birkinshaw, 1996].

12.3 Identification of technical and philosophical issues
Table 21 shows a summary of the key stages in reliability analysis, along with related technical and philosophical issues that have been identified in this report describing the review study.
These issues will form the basis of the development of the framework

<table>
<thead>
<tr>
<th>Stages in reliability analysis</th>
<th>Related technical and philosophical issues</th>
</tr>
</thead>
<tbody>
<tr>
<td>• Structural model</td>
<td>• Uncertainties and relative significance</td>
</tr>
<tr>
<td>• Loading model</td>
<td>• Better quantification and/or reduction of uncertainties</td>
</tr>
<tr>
<td>• Failure modes</td>
<td>• Compatibility of accuracy of sub-models</td>
</tr>
<tr>
<td>• Failure criteria</td>
<td>• Validation of methods in part/full (experiments, benchmarking, performance)</td>
</tr>
<tr>
<td>• Limit states</td>
<td>• Setting target reliability</td>
</tr>
<tr>
<td>• Uncertainties in loading and resistance</td>
<td>• Criteria for consistency in assessments</td>
</tr>
<tr>
<td>• Structural resistance prediction</td>
<td>• Criteria for interpreting as absolute values in decision making</td>
</tr>
<tr>
<td>• System effects</td>
<td>• Lessons from other industries (on consistency, interpretation, values, etc.)</td>
</tr>
<tr>
<td>• Reliability methods</td>
<td>• Human factors relating to competence and guidance for users</td>
</tr>
<tr>
<td>• Uncertainties and sensitivities</td>
<td>• Integration with design or re-assessment process</td>
</tr>
<tr>
<td>• Computer programs/tools</td>
<td>• Integration with other hazards and overall hazard management systems</td>
</tr>
</tbody>
</table>

Table 21: Main issues to be addressed in the development of a generic framework
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