Structural system reliability framework for fixed offshore platforms

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ABSTRACT

This research has arisen from recent changes in attitudes to offshore safety. To help demonstrate structural safety to the Health and Safety Executive, operators are increasingly using structural system reliability assessments. While significant developments have been achieved in this area, there are still a number of uncertainties associated with such assessments. Unresolved technical issues also introduce significant variability in the results. The aim of this project is to develop a framework for system reliability, which will set a basis for moving towards more consistent reliability assessments.

An extensive review study was undertaken first to establish the state of the art in the area of structure system reliability analysis of offshore structures. Based on the findings of this study, a generic system reliability framework was developed which was then developed further for specific application to fixed offshore platforms. These initial studies identified some of the key technical issues that required further investigation. The subsequent offshore application and sensitivity study, using a representative fixed platform model, concentrated on these issues and in particular on the effects of foundation parameters on ultimate strength and their interaction with other key parameters in determining the resistance function.

The effect of foundation parameters and different modelling methods on system strength and reliability of fixed offshore platforms, which has largely been neglected in the past, was also investigated. The response surface methodology was developed for system reliability assessment of offshore structures incorporating the effect of foundation reliability. The findings were then used to revise the framework and provide more comprehensive account of key steps in the process of system reliability assessment. Some guidelines on the application of the response surface technique to fixed platform assessment were developed. In addition, an initial screening tool was also proposed for assessing the level of complexity required for the resistance model of the reliability assessment.

The presentation of the reliability framework provides a comprehensive account of the various steps, methods and decisions associated with system reliability analysis. The framework, which can be used in both the design and reassessment of structures, can provide a basis for moving towards more consistent reliability assessments. Recommendations on areas that require further research are also presented.
TABLE OF CONTENTS

List of tables .............................................................................................................. v
List of figures ............................................................................................................... vii
Acknowledgements .................................................................................................... x
Notation ....................................................................................................................... xi
Terminology ................................................................................................................... xiii

Chapter 1. INTRODUCTION .................................................................................... 1
  1.1 Offshore installations ......................................................................................... 1
  1.2 The UK offshore industry .................................................................................... 3
  1.3 Attitudes to safety ............................................................................................... 3
  1.4 Structural reliability ........................................................................................... 4
  1.5 Aims and objectives ........................................................................................... 6
  1.6 Scope of work ...................................................................................................... 7
  1.7 Layout of thesis .................................................................................................. 7

CHAPTER 2. REVIEW STUDY .................................................................................. 9
  2.1 Introduction .......................................................................................................... 9
  2.2 Generic reliability issues ...................................................................................... 10
    2.2.1 Uncertainty ..................................................................................................... 10
    2.2.2 Sensitivity ....................................................................................................... 11
    2.2.3 Distribution ..................................................................................................... 11
    2.2.4 Structural modelling ....................................................................................... 11
    2.2.5 Confidence ..................................................................................................... 13
    2.2.6 Improving consistency ................................................................................... 14
    2.2.7 Targets for reliability ..................................................................................... 14
    2.2.8 Lessons from other industries ....................................................................... 15
    2.2.9 Guidelines ...................................................................................................... 17
  2.3 Uncertainty and sensitivity .................................................................................. 17
    2.3.1 Qualitative descriptions of uncertainty ............................................................ 17
    2.3.2 Mathematical description of uncertainty ........................................................ 19
    2.3.3 Modelling uncertainty .................................................................................... 22
    2.3.4 Sensitivity studies ......................................................................................... 23
    2.3.5 Generic uncertainty and sensitivity considerations ......................................... 26
  2.4 Structural Models ............................................................................................... 27
    2.4.1 Advancement of computer models ................................................................. 27
    2.4.2 Specific examples of models and analyses undertaken using different software 28
  2.5 Environmental parameters ............................................................................... 34
    2.5.1 Waves ............................................................................................................ 35
    2.5.2 Current ........................................................................................................... 40
    2.5.3 Wind ............................................................................................................... 41
    2.5.4 Extreme environmental event methodologies ................................................. 42
    2.5.5 Joint probability of wind, wave and current occurrence .................................. 43
    2.5.6 Environmental uncertainties and sensitivities .............................................. 47
    2.5.7 The 'airgap' issue ......................................................................................... 50
  2.6 Foundation Modelling ....................................................................................... 50
    2.6.1 Probabilistic foundation modelling methods .................................................. 52
    2.6.2 Deterministic foundation modelling methods ............................................... 57
    2.6.3 Foundation uncertainty ................................................................................ 61
    2.6.4 Generic foundation considerations ............................................................... 61
  2.7 Ultimate Capacity Predictions .......................................................................... 62
    2.7.1 Different approaches to ultimate capacity prediction ................................... 63
    2.7.2 Benchmarking studies .................................................................................. 65
    2.7.3 Generic conclusions regarding ultimate capacity prediction ....................... 69
### Structural System Reliability Framework For Fixed Offshore Platforms

#### CHAPTER 3. INITIAL FRAMEWORK DEVELOPMENT

<table>
<thead>
<tr>
<th>Subchapter</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.1 Introduction</td>
<td>98</td>
</tr>
<tr>
<td>3.1.1 Background</td>
<td>98</td>
</tr>
<tr>
<td>3.1.2 Need for a more rational approach to structural reliability analysis</td>
<td>98</td>
</tr>
<tr>
<td>3.1.3 Identification of technical and philosophical issues</td>
<td>99</td>
</tr>
<tr>
<td>3.2 Initial development of generic framework</td>
<td>99</td>
</tr>
<tr>
<td>3.2.1 Top-level flowchart framework</td>
<td>99</td>
</tr>
<tr>
<td>3.2.2 Detailed generic flowchart framework</td>
<td>102</td>
</tr>
<tr>
<td>3.2.3 Tabular framework</td>
<td>107</td>
</tr>
<tr>
<td>3.3 Framework specific to the design of fixed offshore platforms</td>
<td>109</td>
</tr>
<tr>
<td>3.4 Different reliability assessment methods</td>
<td>113</td>
</tr>
<tr>
<td>3.4.1 Minimal analysis approach</td>
<td>114</td>
</tr>
<tr>
<td>3.4.2 Response surface technique</td>
<td>115</td>
</tr>
<tr>
<td>3.4.3 Numerical simulation approach</td>
<td>116</td>
</tr>
<tr>
<td>3.4.4 System analysis approach</td>
<td>118</td>
</tr>
<tr>
<td>3.5 Discussion of the initial framework</td>
<td>120</td>
</tr>
<tr>
<td>3.6 Identification of areas of focus for more detailed studies</td>
<td>122</td>
</tr>
<tr>
<td>3.6.1 The effect of individual parameters</td>
<td>122</td>
</tr>
<tr>
<td>3.6.2 The effect of key parts of the process</td>
<td>123</td>
</tr>
<tr>
<td>3.6.3 Analysis of different methods of reliability assessment</td>
<td>123</td>
</tr>
</tbody>
</table>

#### CHAPTER 4. SENSITIVITY STUDY OF INDIVIDUAL PARAMETERS

<table>
<thead>
<tr>
<th>Subchapter</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.1 Introduction</td>
<td>124</td>
</tr>
<tr>
<td>4.1.1 Background</td>
<td>124</td>
</tr>
<tr>
<td>4.1.2 Scope of work</td>
<td>124</td>
</tr>
<tr>
<td>4.2 Procedure adopted for structural analysis assessments</td>
<td>125</td>
</tr>
<tr>
<td>4.2.1 Introduction</td>
<td>125</td>
</tr>
<tr>
<td>4.2.2 Loading application</td>
<td>127</td>
</tr>
<tr>
<td>4.2.3 Structural idealisation</td>
<td>128</td>
</tr>
<tr>
<td>4.2.4 Solution derivation</td>
<td>130</td>
</tr>
<tr>
<td>4.2.5 Code checking of joints</td>
<td>131</td>
</tr>
<tr>
<td>4.2.6 Post-processing of results</td>
<td>132</td>
</tr>
<tr>
<td>4.2.7 Failure assessment and criteria</td>
<td>132</td>
</tr>
<tr>
<td>4.3 Yield strength parametric study</td>
<td>134</td>
</tr>
<tr>
<td>4.3.1 Background information</td>
<td>134</td>
</tr>
<tr>
<td>4.3.2 Methodology</td>
<td>134</td>
</tr>
<tr>
<td>4.3.3 Summary of results</td>
<td>134</td>
</tr>
<tr>
<td>4.3.4 Examination of results</td>
<td>135</td>
</tr>
</tbody>
</table>
4.3.5 Plastic hinge locations ................................................................. 137
4.3.6 Pile utilisation at ultimate load .................................................. 139
4.3.7 Discussion of results ................................................................. 142
4.3.8 Results from other research ..................................................... 145
4.4 Foundation parametric study ......................................................... 145
  4.4.1 Background information .......................................................... 145
  4.4.2 Methodology ............................................................................ 146
  4.4.3 Summary of results ................................................................. 148
  4.4.4 Examination of results ............................................................ 149
  4.4.5 Plastic hinge locations .............................................................. 150
  4.4.6 Pile utilisation at ultimate load ................................................ 152
  4.4.7 Discussion of results ............................................................... 155
4.5 Concluding remarks ..................................................................... 159

CHAPTER 5. THE EFFECT OF KEY PARTS OF THE PROCESS: FOUNDATION ASSESSMENT ................................................. 160
5.1 Introduction .................................................................................. 160
  5.1.1 Scope of work ........................................................................ 161
  5.1.2 Assessment of soil types in the North Sea ............................... 162
  5.1.3 Foundation capacity assessment methods .............................. 164
5.2 The effect of soil profile and assessment method for piles in sand .................................................................................. 166
  5.2.1 Introduction ............................................................................ 166
  5.2.2 Case studies using the IC method .......................................... 168
  5.2.3 Case studies using the API method ...................................... 169
  5.2.4 Analysis cases run ................................................................. 169
  5.2.5 Results .................................................................................. 173
  5.2.6 Discussion of results ............................................................. 176
5.3 The effect of cyclic loading and ageing for piles in sand .................. 177
  5.3.1 Introduction ............................................................................ 177
  5.3.2 Effect of cyclic loading on piles in sand ................................. 177
  5.3.3 Effect of ageing on piles in sand ........................................... 178
  5.3.4 Analysis cases run ................................................................. 178
  5.3.5 Results .................................................................................. 179
  5.3.6 Discussion of results ............................................................. 183
5.4 Further investigations into parameter sensitivity ............................ 184
5.5 Concluding remarks .................................................................... 187

CHAPTER 6. SYSTEM RELIABILITY ANALYSIS METHODS ............................... 190
6.1 Introduction ................................................................................ 190
  6.1.1 Basis for study ..................................................................... 190
  6.1.2 Scope of work ..................................................................... 191
6.2 Procedure adopted for reliability analysis .................................... 192
  6.2.1 Introduction ....................................................................... 192
  6.2.2 Probabilistic analysis ......................................................... 192
6.3 Application of the ‘minimal’ analysis approach in this research ........ 193
  6.3.1 Methodology .................................................................... 193
  6.3.2 Summary of intermediate results ........................................ 193
  6.3.3 Derivation of reliability index and probability of failure ......... 195
  6.3.4 Comparison with results from other investigators ............... 198
6.4 Application of the response surface technique in this research ........ 201
  6.4.1 Methodology .................................................................... 201
  6.4.2 Summary of intermediate results ........................................ 204
  6.4.3 Derivation of reliability index and probability of failure ......... 207
  6.4.4 Other relevant investigations ............................................. 208
6.5 Report on third party application of the system analysis approach ................................................................. 209
6.5.1 Methodology ............................................................................................................................................ 209
6.5.2 Summary of intermediate results .......................................................................................................... 210
6.5.3 Derivation of reliability index and probability of failure ........................................................................... 210
6.6 Overall comparison of reliability analysis methods ............................................................................................. 211
6.6.1 Loading ..................................................................................................................................................... 211
6.6.2 Resistance .................................................................................................................................................. 213
6.6.3 Reliability .................................................................................................................................................... 214
6.6.4 Comparison of results ................................................................................................................................. 215
6.7 Preliminary development of a simplified system analysis approach ...................................................................... 216
6.7.1 Simplified system analysis methods in published works ............................................................................... 216
6.7.2 Background to ideas for new initial screening tool ..................................................................................... 219
6.7.3 Proposed basis of assessment .................................................................................................................... 221
6.7.4 Limits of applicability ................................................................................................................................ 225
6.8 Concluding remarks ...................................................................................................................................... 225

CHAPTER 7. FINAL FRAMEWORK ....................................................................................................................... 228
7.1 Introduction and background .......................................................................................................................... 228
7.2 Presentation of updated framework ................................................................................................................ 228
7.2.1 Top-level framework .................................................................................................................................. 228
7.2.2 Second-level framework ............................................................................................................................ 229
7.2.3 Presentation of detailed frameworks for each stage ..................................................................................... 233
Numerical simulation technique .................................................................................................................................. 251
7.3 Benefits and potential applications of the framework ......................................................................................... 257
7.3.1 Moving towards "true" reliability .............................................................................................................. 257
7.3.2 Improved preparation ................................................................................................................................. 257
7.3.3 Improved consistency ................................................................................................................................. 258
7.3.4 Communication tool .................................................................................................................................. 258
7.3.5 Application tool ......................................................................................................................................... 258
7.3.6 Management tool .................................................................................................................................... 258
7.3.7 Quality assurance tool ............................................................................................................................... 258
7.3.8 Education or training tool ......................................................................................................................... 258
7.3.9 Potential usefulness of framework within project lifecycle ........................................................................ 259
7.4 Concluding remarks ...................................................................................................................................... 259

CHAPTER 8. CONCLUSIONS .................................................................................................................................... 262
8.1 Introduction ...................................................................................................................................................... 262
8.2 Review of system reliability assessment ......................................................................................................... 263
8.2.1 Summary of generic issues relating to offshore structures .......................................................................... 263
8.2.2 Summary of issues specific to fixed steel platform type offshore structures ............................................. 264
8.3 Structural system reliability framework development ....................................................................................... 266
8.4 Investigation into key parameters .................................................................................................................. 268
8.5 Investigation into key parts of the process – assessment of foundations ......................................................... 269
8.6 Reliability assessment methods ..................................................................................................................... 271
8.7 Final structural system reliability framework .................................................................................................. 274
8.8 Original work and contribution to knowledge .................................................................................................. 275
8.9 Areas for further work .................................................................................................................................. 276

REFERENCES ..................................................................................................................................................... 278
List of tables

Table 1: Non-linear software for pushover analysis of offshore structures [Bolt et al., 1995] ..................... 12
Table 2: 50-year extreme storm condition for Lomond platform for W direction [Brown and Root, 1993b] ................................................................. 29
Table 3: Conditions studied for the Lomond platform ................................................................................. 30
Table 4: 50-year extreme storm conditions for Montrose platform Northerly direction ........................................ 33
Table 5: Conditions studied for the Montrose platform ............................................................................... 33
Table 6: Results obtained for the Montrose Platform .................................................................................. 34
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 Beiboor, 1990] .................................................................................................................. 44
Table 8: Parameters of lognormal distribution for 20 and 100 year loads in various geographical areas [Efthymiou et al., 1997] ............. 45
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] ................................................................. 46
Table 10: Summary of criteria, procedures and practice/reference norms in deterministic and probabilistic approaches [Lloyd, 1985] ................................................................. 47
Table 11: Assessment of capacity predictions for piles in sand [Jardine and Chow, 1996b] .................... 60
Table 12: Assessment of peak clay shaft capacity prediction [Jardine and Chow, 1996b] .................... 60
Table 13: The 50-year extreme storm conditions for Lomond platform (W direction) ......................... 79
Table 14: Results for intact and damaged structure for design and extreme environmental loading conditions for Lomond platform (W direction) ................................................................. 80
Table 15: Reliability results for intact and damaged Lomond platform (W. direction) ........................................ 80
Table 16: The 50-year extreme storm conditions for Leman 49/27 AP (N direction) ........................................ 82
Table 17: Conditions studied for the Leman 49/27 AP platform .................................................................. 82
Table 18: Results obtained for the intact and damaged Leman 49/27 AP platform ........................................ 83
Table 19: Summary of case studies and the effects examined ........................................................................ 85
Table 20: Summary of probability equations used ...................................................................................... 92
Table 21: Main issues to be addressed in the development of a generic framework ........................................ 97
Table 22: Standard flow chart symbols used .............................................................................................. 100
Table 23: Summary outline generic framework presented in tabular format .......................................... 108
Table 24: Detailed breakdown table for Stage 2: Modelling of structure ................................................ 109
Table 25: Framework Stage 2. Modelling of structure ........................................................................... 111
Table 26: Summary outline framework specific to design of fixed offshore platforms ........................................ 112
Table 27: Summary of runs performed for the yield strength parameter study (*peak not reached) .......... 135
Table 28: Plastic hinges recorded for the yield strength parameter study (*peak not reached) ................. 137
Table 29: First 5 elements where plastic hinges (PH) were recorded in yield strength study .................... 138
Table 30: Summary of runs performed for the yield strength parameter study (*peak not reached). ....... 142
Table 31: Peak load factor results for foundation stiffness and capacity parameter study .................... 148
Table 32: Runs performed in the foundation capacity parameter study .................................................. 149
Table 33: Plastic hinges recorded for the foundation capacity parameter study ........................................ 151
Table 34: First 5 elements where plastic hinges were recorded for cases in the foundation study .......... 152
Table 35: Peak load factor exhibited for foundation capacity parameter study runs ........................................ 155
Table 36: Cases to be studied to investigate the effect of changing the soil profile ........................................ 169
Table 37: Peak load factor results for different profiles for IC and API methods ........................................ 173
Table 38: Percentage of plastic hinges formed up to peak load in each location ........................................ 175
Table 39: Description of the first set of cases to be studied to investigate the effect of ageing and cyclic loading for jacket-dominated failure of the structure .......................................................... 179
Table 40: Description of second set of cases to be studied to investigate the effect of ageing and cyclic loading for mixed mode failure of the structure .................................................. 179
Table 41: Ageing and cyclic loading effect results for jacket-dominated failure of structure ............... 180
Table 42: Percentage difference results for the effect of ageing and cyclic loading for the first set of analyses based on jacket-dominated failure ............................................................... 181
Table 43: Peak load factor results for ageing and cyclic loading effect runs, for mixed failure mode of the structure ................................................................................................................. 181
Table 44: Percentage difference results for the effect of ageing and cyclic loading for the first set of analyses based on jacket-dominated failure ............................................................... 181
Table 45: Summary of runs performed for the foundation stiffness study based on the IC assessment method and 'design' profile I ................................................................. 183
Table 46: Summary of analyses performed to investigate the effect on peak load factor of changing the foundation capacity .......................................................................................... 185
Table 47: Design and ultimate base shear results derived using SAFJAC results for Leman AP ......... 194
Table 48: Overall probability of failure for Leman AP from reliability analyses derived in this research ........................................................................................................................................... 197
Table 49: Load and resistance parameters used in the comparison of minimal analyses .................. 198
Table 50: 50-year design conditions used in SAFJAC and RASOS analyses ..................................... 198
Table 51: Ultimate base shear values derived from SAFJAC and RASOS analyses ............................ 199
Table 52: Reliability analysis results, using lognormal distributions, using different methods, for four wave approach directions .................................................................................. 200
Table 53: Peak load factor results and predictions from 29 observation points ......................... 205
Table 54: Set up used in the reliability analysis in PROBAN .............................................................. 207
Table 55: Results obtained from PROBAN for the design point from the response surface analysis (using 5th order polynomial equation) .............................................................. 207
Table 56: Difference in prediction and measured peak load factor at the design point .................... 208
Table 57: Design and ultimate base shear results from the system analysis approach ..................... 210
Table 58: Stochastic model for basic load parameters used by WSAtkins [WSAtkins, 1997b] ............. 211
Table 59: Stochastic model for basic resistance parameters used in RASOS analyses ..................... 211
Table 60: Reliability results for Leman AP for wave from platform West [WSAtkins, 1997b] ...... 211
Table 61: Key reliability results for Leman AP derived by different methods ................................. 216
Table 62: Reliability results for foundation only failure sensitivity study ......................................... 224
Table 63: Reliability results for jacket only failure sensitivity study .............................................. 224
Table 64: Summary outline framework presented in a tabular format ........................................... 232
Table 65: Detailed breakdown table Stage 1: Assessment of platform conditions ......................... 234
Table 66: Detailed breakdown table Stage 2: Modelling of structure ............................................ 236
Table 67: Detailed breakdown table Stage 3: Environmental load assessment ............................... 237
Table 68: Detailed breakdown table Stage 4b: foundation capacity assessment for sands ............. 243
Table 69: Detailed breakdown table Stage 4a: Foundation capacity assessment for clay soils ........ 245
Table 70: Detailed breakdown table Stage 5: System analysis model derivation ............................. 245
Table 71: Detailed breakdown table Stage 6a: Reliability analysis: 'component' based approach .... 247
Table 72: Sub-table Stage 6a Part 1: Reliability analysis using 'component' based approach using the 'minimal' analysis technique ................................................................. 253
Table 73: Sub-table Stage 6a Part 3: Reliability analysis using 'component' based approach using the response surface technique ................................................................. 254
Table 74: Sub-table Stage 6a Part 3: Reliability analysis using 'component' based approach using the numerical simulation technique ................................................................. 255
Table 75: Stage 6b: Reliability analysis using 'system' based approach .......................................... 257
List of figures

Figure 1: Typical load effects acting on a platform .............................................................. 1
Figure 2: Diagram showing risk “initiators” [Crawley et al., 1995] ........................................... 2
Figure 3: Diagram showing the hazards encountered in offshore structures ............................. 2
Figure 4: Uncertainties in reliability assessment [Melchers, 1999] ............................................. 5
Figure 5: “Generic decision framework” [Nicholls, 1997] .......................................................... 16
Figure 6: Diagram showing the API procedure for calculation of wave plus current forces for static
analysis [API, 1993b] .................................................................................................................. 40
Figure 7: Top-level generic framework flowchart ................................................................. 101
Figure 8: Generic framework for both new (design) and old (reassessment) structures ............. 103
Figure 9: Generic framework for design and reassessment of structures - part 1 ....................... 104
Figure 10: Generic framework for design and reassessment of structures - part 2 ...................... 105
Figure 11: Generic framework for design and reassessment of structures - part 3 ....................... 106
Figure 12: Generic framework for design and reassessment of structures - part 3 ....................... 107
Figure 13: Specific framework for design of fixed offshore platforms ..................................... 110
Figure 14: Generic framework - reliability assessment extract .................................................. 113
Figure 15: Framework extract showing steps involved in the “minimal” analysis approach .......... 114
Figure 16: Framework extract showing steps involved in the response surface method ............... 116
Figure 17: Framework extract showing steps involved in the numerical simulation approach ...... 117
Figure 18: Framework extract showing steps involved in the “system” analysis approach .......... 119
Figure 19: Simplified representation of the structural analysis process [Holzer, 1985] ............... 125
Figure 20: SAFJAC model of Leman AP platform, showing location of nodes ......................... 126
Figure 21: Diagram showing pile layout for Leman AP platform with true and platform North ..... 127
Figure 22: Replacement of strain hardening plasticity by an ideal bi-linear elastic-plastic
approximation, where: \( \sigma = \) stress and \( \epsilon = \) strain ................................................................. 128
Figure 23: Generic five-part curve used to describe the force-displacement and moment-rotation of
type 41 elements in SAFJAC [BOMEL, 1998] .............................................................................. 130
Figure 24: Load - displacement results for pushover analyses, with different values of yield stress, for
the wave approach direction from platform North ................................................................. 136
Figure 25: Extract of load - displacement results for the wave approach direction from platform North,
for load factor greater than 1.5 .............................................................................................. 136
Figure 26: Plastic hinges formed in the foundation piles in the yield strength study ..................... 137
Figure 27: Side elevations of Rows 1 and 2 of Leman AP, showing location of elements where the
plastic hinges occurred, for cases run within the yield strength study ................................. 139
Figure 28: Pile utilisation for an 8% reduction in yield strength (wave from platform North) ....... 140
Figure 29: Pile layout for Leman AP showing wave approach direction ...................................... 140
Figure 30: Pile utilisation plot for Pile A Row 2 for case with yield strength reduced by 5% ....... 141
Figure 31: Pile utilisation plot for Pile A Row 2 for original case (yield strength unchanged) ....... 141
Figure 32: Pile utilisation plot for Pile A Row 2 with yield strength increased by 5% .................... 142
Figure 33: Yield strength against peak load factor exhibited ..................................................... 143
Figure 34: Deformation (m) at peak load for case with yield strength increased by 10% ............... 144
Figure 35: Deformation (m) at peak load for case with yield strength decreased by 10% ............. 144
Figure 36: Axial T-z data for group 3 elements in the model ...................................................... 147
Figure 37: Lateral P-y data for group 3 elements in the model .................................................... 147
Figure 38: Load factor vs. displacement for analyses performed for the foundation stiffness and
capacity parameter study ....................................................................................................... 149
Figure 39: Typical load-displacement plots for the three different failure modes ....................... 150
Figure 40: Number of plastic hinges formed in piles, within the foundation capacity study ..........151
Figure 41: Side elevations of Rows 1 and 2 of Leman AP, showing location of elements where the first five plastic hinges occurred, for cases run within the foundation study ..........152
Figure 42: Pile utilisation plot for Pile A Row 2 for case where foundation capacity multiplied by a factor of 0.7 ..........................................................................................................................153
Figure 43: Pile utilisation plot for Pile A Row 2 for the original foundation case ..........154
Figure 44: Pile utilisation plot for Pile A Row 2 for foundation capacity x1.3 ..........154
Figure 45: Change in peak load factor with foundation capacity for analyses performed for the foundation capacity parameter study ..........156
Figure 46: Dominant failure modes for peak load factor with foundation capacity ..........156
Figure 47: Deformation (m) at peak load for a sample foundation dominated failure ..........157
Figure 48: Deformation (m) at peak load for a sample mixed mode failure ..........157
Figure 49: Deformation (m) at peak load for a sample jacket dominated failure ..........158
Figure 50: Pile-supported steel jackets platforms installed in the North Sea in different soil types [after Bond et al., 1997] ..........163
Figure 51: Cone CPT (MPa) with depth below top of clay layer (m) for the four soil profiles derived ..........167
Figure 52: Total axial pile capacity (corrected for scour) for the eight cases studied for compression loading (data supplied by IC) ..........170
Figure 53: Total axial pile capacity (corrected for scour) for the eight cases studied for tension loading (data supplied by IC) ..........170
Figure 54: Maximum shear stress against depth of pile (m) for API assessed cases 1, 2 and 3 for compression loading (data supplied by IC) ..........171
Figure 55: Maximum shear stress against depth of pile (m) for API (North Sea variant) assessed cases 1, 2 and 3 for tension loading (data supplied by IC) ..........171
Figure 56: Maximum shear stress against depth of pile (m) for IC assessed cases for compression loading (data supplied by IC) ..........172
Figure 57: Maximum shear stress against depth of pile (m) for IC assessed cases for tension loading (data supplied by IC) ..........172
Figure 58: Load-deflection characteristics of analyses performed using the API method (the ‘original’ run here was that initially used from data supplied in the model) ..........174
Figure 59: Load-deflection characteristics of analyses performed using the IC method (The ‘original’ run here was that initially used from data supplied in the model) ..........174
Figure 60: Plastic hinges formed up to peak load, grouped by assessment method ..........175
Figure 61: Load deflection characteristics exhibited for the ageing and cyclic analyses based on jacket-dominated failure ..........180
Figure 62: Change in peak load factor against change in foundation capacity for the four ageing and cyclic analyses based on jacket-dominated failure ..........181
Figure 63: Load deflection characteristics exhibited by the four ageing and cyclic analyses, based on a mixed mode base case (IC x0.7) ..........182
Figure 64: Change in peak load factor against change in foundation capacity for the four ageing and cyclic analyses based on mixed mode failure ..........182
Figure 65: Load - displacement results for pushover analyses using the IC assessment method with different values of foundation stiffness ..........185
Figure 66: Peak load factor exhibited for runs with different foundation capacity ..........187
Figure 67: Compass plots of 50-year design base shear and ultimate base shear for Leman AP ..........194
Figure 68: Reliability index results obtained by different methods, using lognormal load with COV=0.3, for wave approaching from Platform North ..........197
Figure 69: Bar chart showing design and ultimate base shear results from SAFJAC\(^1\) and RASOS analyses ..........199
Figure 70: Bar chart showing comparison of reliability index values derived using different methods, for four wave approach directions ..........201
Figure 71: Location of the nine runs performed for the CCD...............................204
Figure 72: Predicted peak load factor divided by measured peak load factor against yield strength of the deck columns, jacket legs and braces.................................................206
Figure 73: Predicted peak load factor divided by measured peak load factor against foundation pile capacity ............................................................................................................206
Figure 74: Diagram showing method by which Bea subdivided the platform structure..........................................................218
Figure 75: Load deflection characteristics for three pushover analysis cases studied with different failure modes..........................................................................................................................218
Figure 76: Trendline curve of peak load factor against foundation capacity..............................................................................................220
Figure 77: Foundation capacity against peak load factor for *jacket-dominated* failure ...........................................................................222
Figure 78: Foundation capacity against peak load factor for *foundation-dominated* failure ........................................................................222
Figure 79: Foundation capacity against peak load factor for *mixed-mode* failure scenario ...........................................................................222
Figure 80: Peak load factor against foundation capacity showing linear representations of foundations and jacket dominated failures..............................................................................................223
Figure 81: Top-level generic framework flowchart.........................................................................................................................229
Figure 82: Second-level generic framework flowchart...................................................................................................................230
Figure 83: Detailed generic framework flowchart - assessment of platform conditions .................................................................................233
Figure 84: Extract from second-level generic framework flowchart Stage 2 modelling of structure .................................................................235
Figure 85: Detailed generic framework flowchart - environmental load assessment ...................................................................................237
Figure 86: Sub-framework flowchart – foundation capacity assessment for sands part 1........................................................................239
Figure 87: Sub-framework flowchart – foundation capacity assessment for sands part 2........................................................................241
Figure 88: Sub-framework flowchart – foundation capacity assessment for clays ...............................................................................244
Figure 89: Detailed generic framework flowchart – reliability 'component' assessment .............................................................................246
Figure 90: Sub-framework flowchart – reliability assessment by minimal analysis technique ...................................................................248
Figure 91: Sub-framework flowchart: reliability assessment by response surface technique 1...........................................................................249
Figure 92: Sub-framework flowchart: reliability assessment by response surface technique – part 2...........................................................250
Figure 93: Sub-framework flowchart – reliability assessment by numerical simulation ...................................................................................252
Figure 94: Detailed generic framework flowchart – reliability 'system' assessment .................................................................................................256
Figure 95: Representation of the potential usefulness of the framework at each project phase (Shaded areas represent perceived relevance) ........................................................................259
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Special thanks go to all my fellow researchers in the department, for their friendship and interesting discussions.
Notation

$\beta$  reliability index
$\beta_{LI}$ reliability index derived for the first failure of any member
$\beta_{mnfa}$ reliability index for first failure of a member previously identified as critical
$\beta_{sys}$ reliability index for system failure
$\delta$ interface friction angle between soil and pile wall
$\delta_R$ coefficient of variation of the capacity
$\mu$ mean
$\eta$ error introduced by lack-of-fit approximations.
$\lambda_0$ collapse load factor in direction $\theta$
$\lambda_p$ design level load factor
$\lambda_{pmax}$ ultimate load factor
$\rho$ density
$\sigma_{vo}^\prime$ initial vertical stresses
$\sigma_{rf}^\prime$ radial effective stress at point of shaft failure
$\sigma_{rf}^\prime$ radial effective stress at point of shaft failure
$\sigma_y$ yield strength
$\sigma_v^\prime$ original vertical effective stress in the ground
$\tau_f$ peak local shear stress
$\tau_{fmax}$ limiting value of shaft friction
$\tau_{RZ}$ ultimate skin friction
$\tau_s$ ultimate shear resistance of a single pile
$\Phi$ standard normal distribution
$a$ water particle acceleration
$A_p$ gross end area of pile
$A_s$ side surface area of pile
$C$ current
$C_D$ drag coefficient
$C_M$ inertia coefficient
$C_S$ shape coefficient
CNS Central North Sea
COV coefficient of variation
DTR damage tolerance ratio
$f$ unit skin friction capacity
$f(\theta)$ probability distribution of a parameter, taking a value of $\theta$ before new data
$f(\theta | \text{data})$ probability distribution of $\theta$ afterwards
$f_{ag}$ factor due to soil ageing
$f_c$ foundation capacity
$f_d$ factor due to pile design conservatism
$f_i$ factor due to structural interaction
$f_i^l$ probability density function of pile load
$f_r$ factor due to (load) rate effects
\[ f_R \] probability density function of the capacity or resistance
\[ f_s \] factor due to sampling effects
\[ F_L \] cumulative probability density function of pile load
\[ F_{SC} \] system capacity of the structure
\[ g \] state function
\[ g(X,p) \] limit state
\[ G \] wave-to-wave force uncertainty, due to variations in drag, shielding etc.
\[ HAT \] highest astronomical tide
\[ H_s \] four times the standard deviation of ocean surface elevation.
\[ K \] dimensionless coefficient of lateral earth pressure
\[ L \] load function
\[ L_\theta \] environmental load in direction \( \theta \)
\[ LAT \] lowest astronomical tide
\[ NNS \] Northern North Sea
\[ P(data | \theta) \] conditional probability of having observed the new data, were \( \theta \) true.
\[ P_f \] or \( P(f) \) probability of failure,
\[ P_f(F) \] probability of failure of the foundations
\[ P_f(J) \] probability of failure of the jacket
\[ P_f(sys) \] probability of system failure
\[ q \] unit end bearing capacity
\[ Q \] achieved capacity
\[ Q_a \] actual pile bearing capacity
\[ Q_c \] initial design capacity
\[ Q_f \] skin friction resistance
\[ Q_p \] total end bearing
\[ Q_s \] shaft capacity
\[ R \] resistance
\[ R_u \] ultimate lateral load capacity of platform
\[ RF \] redundancy factor
\[ RIF \] residual resistance factor
\[ RSR \] reserve strength ratio
\[ S \] loading
\[ S_{ref} \] reference base shear force
\[ S_R \] “reference” lateral loading.
\[ S(\omega) \] ocean surface energy spectrum
\[ SNS \] Southern North Sea
\[ t \] time of assessment (up to a maximum of five years)
\[ \tan \delta_f \] interface angle of friction at failure
\[ T_s \] return period in years associated with reference environmental lateral loading
\[ u \] water particle velocity
\[ X \] variable
\[ y \] ‘true’ response
\[ \hat{y} \] estimated response
\[ Z_{\text{seastate}} \] vector of random variables of the seastate
\[ Z_{\text{structure}} \] vector of random variables of the structure
\[ Z_{\text{wave-forces}} \] vector of random variables of the wave-forces
## Terminology

<table>
<thead>
<tr>
<th>Term</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>Availability</td>
<td>The probability that an item is in the working state.</td>
</tr>
<tr>
<td>Damage</td>
<td>The loss of inherent quality suffered by an entity.</td>
</tr>
<tr>
<td>Deterministic method</td>
<td>Calculation method in which basic variables are treated as non-random.</td>
</tr>
<tr>
<td>Durability</td>
<td>Similar to availability, it is the ability of the structure and structural elements to maintain adequate performance in time.</td>
</tr>
<tr>
<td>Failure</td>
<td>A short fall between performance and standards; the generation of undesirable side effects, or neglect of an opportunity. It is multi-causal and rises from complex interactive and independent prior activities.</td>
</tr>
<tr>
<td>Failure rate</td>
<td>The mean number of failures of an item in a given time.</td>
</tr>
<tr>
<td>Failure type</td>
<td>Failure can have various forms including catastrophic or minor, overwhelming or partial, rapid or slow.</td>
</tr>
<tr>
<td>Hazard</td>
<td>A situation that in particular circumstances could lead to harm.</td>
</tr>
<tr>
<td>Hazard rate</td>
<td>The instantaneous probability of a first and only failure of an item.</td>
</tr>
<tr>
<td>Limit states method</td>
<td>Calculation method in which an attempt is made to prevent the structure attaining certain limit states - i.e. states beyond which the structure no longer satisfies the design (performance) requirements.</td>
</tr>
<tr>
<td>Probabilistic method</td>
<td>Calculation method in which the basic variables are treated as random.</td>
</tr>
<tr>
<td>Reliability</td>
<td>The probability that a device will perform its purpose adequately for the period of time intended and under the operating conditions encountered. Covers safety, serviceability and durability of the structure.</td>
</tr>
<tr>
<td>Resistance</td>
<td>Can be applied to any criterion connected with any limit-state.</td>
</tr>
<tr>
<td>Risk</td>
<td>The probability that a particular adverse event occurs during a stated time period, or results from a particular challenge. Is a combination of an event, the event probability and the event consequence.</td>
</tr>
<tr>
<td>Risk analysis</td>
<td>The process of identification of outcomes, estimation of the magnitude of associated consequences of the outcomes, estimation of the probabilities of the outcomes.</td>
</tr>
<tr>
<td>Risk assessment</td>
<td>Involves risk identification, risk estimation (i.e. frequency and consequence) and risk evaluation (i.e. risk acceptance criteria).</td>
</tr>
<tr>
<td>Risk evaluation</td>
<td>The complex process of determining the significance or value of the identified hazards and estimated risk to those concerned with or affected by the decision. Also includes risk perception and the trade-off between perceived risk and perceived benefits.</td>
</tr>
<tr>
<td>Serviceability</td>
<td>The ability of a structure/structural elements to perform adequately in normal use.</td>
</tr>
<tr>
<td>Structural safety</td>
<td>The capacity of a structure to resist all the actions, and also certain specified accidental phenomena, which it will have to withstand during construction and intended, specified use.</td>
</tr>
</tbody>
</table>
Chapter 1.

INTRODUCTION

1.1 Offshore Installations

Offshore platforms are constructed over oil fields to support the necessary equipment required for controlling and performing the oil extraction process in order to enable oil to be extracted from beneath the seabed. The fundamental design requirement of an offshore platform is that it must satisfy the functional need of support structure for offshore oil and gas operations, and be structurally adequate for both operating and extreme loading. There are a number of different loads that must be taken into account at the design stage. It is necessary to determine the foundation conditions at site by carrying out an in-situ site investigation; predict environmental conditions - wave period, tidal surges, wind speeds, current speeds and potential ice forces and earthquake seismic activity; and assess the associated dead and live loads [Dawson, 1993]. There are also additional loads associated with the installation and erection of the platform that must also be taken into account.

Figure 1 shows the main loads on a platform:

Figure 1: Typical load effects acting on a platform
Numerous studies have been undertaken on the hazards affecting offshore structures. One example is where a methodology for concept risk assessment of offshore developments was developed. As part of this, risk initiators were identified for offshore structures. Figure 2 shows structural failure in the context of other risks that can be encountered offshore [Crawley et al., 1995].

This research has been focused on developing a framework for structural system reliability assessments of fixed offshore structures. The framework developed is only designed to assess one hazard that may affect an offshore structure - that of extreme weather. It will not therefore include offshore hazards including fatigue, earthquake, ship impact etc. In the future, other such frameworks could be developed in order to address these hazards. The hazards that are encountered in the offshore environment can be broadly divided into three main types: ‘natural’ hazards e.g. extreme weather, seismic (earthquake), ice and snow; ‘accidental’ hazards e.g. ship impact, aircraft impact, fire and blast; and lastly ‘internal’ hazards e.g. corrosion and fatigue. Figure 3 shows this in a simple representation.
1.2 The UK offshore industry

The development of oil and gas from the central and northern North Sea dates from the 1970s when the deeper waters of the North Sea, particularly those North East of the Shetland Isles, represented the international industry's most demanding frontier for the application of offshore technology. The basis for most such developments in the 1970s and 80s was fixed production platforms comprising large steel or concrete structures supporting operational processing and drilling equipment, as well as living quarters. The 1980s saw the growing application of subsea production technology, with subsea wells linked to production platforms by flowlines. In the 1990s, floating production systems, offering greater mobility and flexibility of usage, have helped to maintain the momentum of production development for smaller fields against a background of continuing low oil prices. The new oil developments in very deep water West of the Shetland Isles are using this technology. The 1990s also saw a number of the early platforms reaching the end of their intended service life. As a result dutyholders who wished to continue operating those platforms had to perform a complete reassessment in order to ensure that structural integrity and safety were sustained.

Since the UK Offshore Industry began, it has developed considerably in terms of both technical achievements and safety awareness. In 1996 the UK Offshore Operators Association (UKOOA) presented its appraisal of “Thirty years of UK offshore oil and gas exploration and development” [UKOOA, 1996]. It stated that “80% of the total primary fuels produced by Britain is provided; 29,000 people are directly employed offshore; an estimated 300,000 onshore jobs have been created throughout the country, in over 5000 companies, supporting and servicing the offshore industry in both construction and operations.” An approximate value of Britain’s total oil and gas production is in the order of £10.6 billion [UKOOA, 1996]. In the last few years, however, sustained depression of the North Sea oil price has meant that the UK offshore industry is experiencing an increased financial challenge. Strict cost-cutting measures, including shelving of major projects, downsizing of numbers of staff, and company mergers have been implemented as a result. This research has therefore become even more significant, with the industry looking for ways to reduce costs, whilst facing ever more demanding technical challenges.

1.3 Attitudes to safety

In July 1988 an explosion occurred on the production deck of the Piper Alpha platform. A fireball erupted causing a fire to develop, which, along with further explosions and dense
smoke, caused failure of most of the emergency systems on board. Subsequent rupture of the gas risers initiated several major explosions that caused significant structural collapse of the platform. Out of the crew of 226, 165 died - the highest death toll in the history of offshore operations in the North Sea. The subsequent public inquiry, lead by Lord Cullen, produced a detailed report in 1990 [Cullen, 1990] which critically appraised the: working practices on board, owners’ managerial procedures and attitudes to safety, behaviour of the systems for control, shut-down, evacuation and rescue, and inspecting and monitoring roles of the Department of Energy. The Cullen report recommended over 100 changes to every aspect of offshore safety. The four main recommendations were as follows:

- Introduction of a Formal Safety Assessment (FSA) – a demonstration that hazards and their risks to personnel have been identified and reduced as low as reasonably practicable (ALARP).
- Better safety management – an introduction of regular external safety audits.
- Change to regulations – a phasing out of prescriptive guidance, and introduction of performance objectives “goal setting” regime.
- A new regulatory body - a single regulator with the philosophy, management culture and expertise, i.e. the Health and Safety Executive (HSE).

A ‘New Era’ of safety awareness therefore dawned in the early 1990s. This included regulations in 1992 [Offshore Installations (Safety Case) Regulations, 1992] requiring all installations operating in the UK sector to submit a formal safety case to the HSE. The dutyholders were given until 30 November 1995 to have the safety cases formally accepted by HSE - after this date, by law, no installation could continue to operate in the UK sector. The dutyholder is a legal term used to identify the “person responsible for ensuring the law is complied with, and is normally the operator of an installation producing oil or gas” [UKOOA, 1996].

1.4 Structural reliability

As part of the safety case requirements, it is necessary for dutyholders to provide evidence that the offshore platform is structurally robust and is able to withstand the forces from the environment in which it operates. Structural reliability is a ‘tool’ that is often used to provide an assessment of the integrity of a structure under extreme environmental loading. Structural reliability is based on making decisions irrespective of the state of completeness and quality of information, and is formulated under conditions of uncertainty, so that the consequence of a given decision cannot be determined with complete confidence. It is concerned with the rational treatment of uncertainties in structural engineering design and
the associated problems of rational decision making [Frieze, 1989]. Uncertainty in reliability assessments can arise from a number of sources as summarised in Figure 4.

![Decision — Phenomenological — Modelling — Physical — Prediction — Human factors — Statistical](image)

**Figure 4: Uncertainties in reliability assessment [Melchers, 1999]**

Identification of uncertainties for complex systems can be difficult. Phenomenological uncertainty may be considered to arise whenever a form of construction or design technique generates uncertainty about any aspect of the possible behaviour of the structure under construction, service and extreme conditions. It is of particular relevance to 'novel' projects but can only be assessed in subjective terms. Decision uncertainty arises from the decision as to whether a particular phenomenon has occurred, for example, if a limit state violation has occurred. It can be formulated in terms of a probability density function for the uncertain criterion. Modelling uncertainty is associated with the use of simplified relationships between the basic variables used to represent 'real' relationships or phenomena of interest. A number of different types of modelling uncertainty exist, and some aspects of this have been a focus of this research. Physical uncertainty arises from the inherent random nature of a basic variable. Prediction uncertainty arises from the need to involve the prediction of some future state of affairs in the context of a structural reliability assessment. Human factors give rise to uncertainty in terms of human errors and human intervention and their interaction. Statistical uncertainty exists when a statistical estimator such as the sample mean is derived from available data. It can be incorporated into reliability analysis by setting parameters such as the mean and variance as random variables themselves. Alternatively the reliability analysis can be repeated using different values of the parameters to indicate sensitivity.

Investigations into sensitivity involve a study of the effect of each of the different parameters on the 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 small, then the variable can be treated deterministically; but if changes affect the overall reliability significantly, then the variable must be modelled by using the best available distribution.
There is a need within the offshore industry, to move towards a common approach when assessing structural reliability. The use of different models, software and users means variations in methods and assumptions, which implies that different modelling and statistical uncertainties are included in the analysis. There is a need to reduce or better quantify modelling uncertainty, as well as to consider improved means of incorporating modelling uncertainty in reliability analysis. A lack of guidelines or a framework within which such work is undertaken has lead to the development of inconsistent assessments, and other investigators have identified the need for setting guidelines [Onoufriou et al., 1994; Gierlinski et al., 1993]. A framework in which such aspects are included, combined with information on the interpretation of the results, is to be developed within this research [Sigurdsson et al., 1994; Tromans et al., 1993].

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 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 then be used as a measure of safety, or as a useful decision variable.

The probability of failure and reliability index are derived from integration of the probability distributions of load and resistance. These measures can be calculated by a reliability method which can be any amongst several available methods, including approximate analytical methods such as first and second order reliability methods, as well as numerical simulation methods. 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 derivation of the probability of failure.

1.5 Aims and objectives

The main objective of this research 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 true reliability. The perceived benefits of this project include: providing a basis for future working practice/guidance as a move towards consistent reliability, with improved preparation, improved consistency in results; along with allowing the framework to be used as application, management, quality assurance and education/training tools.

It is in the climate of the fully functioning ‘goal-setting’ regime that this project is significant. As mentioned previously, dutyholders are no longer restricted to inflexible guidance, but are now expected to demonstrate safety in terms of stated performance objectives. The framework developed within this project will go towards aiding operators to make improvements to reliability assessments in order to move towards more consistent reliability assessments. It will not become incorporated into strict guidance, but may become part of improved working practices that may be recommended by HSE in the future.

1.6 Scope of work

Of the 460 structures installed in the waters of North West Europe by the offshore petroleum industry, just over half (235) are located in the waters of the UK Continental Shelf. There are five general types of platform in use in the UK sector of the North Sea. These are fixed steel jacket platforms, concrete platforms, semi-submersible rigs, jack-up rigs and other (e.g. monopod, tension leg platforms). Of the 235 installations, over 90% are pile supported steel jacket structures [Bond et al., 1997]. Since the majority of installations is of the fixed steel jacket type, significantly more information is available on analyses performed on jacket type structures than any other type. Thus, the framework developed here is for fixed steel jacket platforms. A more generic version of it could be developed later, which could then be used for the reliability analysis of other types of platform. This research is therefore focused on the structural performance and reliability of fixed steel jacket platforms and their potential exposure to extreme weather conditions.

1.7 Layout of thesis

The general subject of structural reliability assessment is introduced in Chapter 2. Detailed aspects relating to structural system reliability of offshore structures under extreme environmental loading are presented, and relevant studies performed to date are reviewed. A number of case studies where reliability analysis has been undertaken are described. The key philosophical and technical issues that need to be addressed in a structural system reliability analysis are identified. The need for a more consistent approach is examined, and the need to develop a framework is presented.
An initial framework is developed in Chapter 3. This presents development work on a number of different methods for the presentation of a framework. The reliability assessment methods that are currently used in the offshore industry are examined and detailed frameworks describing the activities required are shown. Specific sources of uncertainty are identified, along with those areas where knowledge is lacking. The framework enables identification of areas where further work is needed in order to converge towards more consistent reliability predictions. Three main areas are identified for further study: examination of the effect of certain individual parameters used within the overall reliability analysis process, scrutiny of the effect of key parts of the process and analysis of the different methods of reliability assessment.

A parametric sensitivity study is presented in Chapter 4, focused on two main areas. Firstly, the effect that changes to the yield strength has on ultimate capacity prediction. Secondly, the effect that changes to the stiffness and capacity of the foundations has on the ultimate capacity prediction. The combined effect of these two parameters is also examined. These studies are performed using a detailed platform model in a 3D non-linear finite element software package.

The effect of key parts of the process are then examined in Chapter 5, which focuses on the assessment of foundation capacity. Piles in sand are studied in detail and effects such as cyclic loading and ageing are examined and quantified.

The different system reliability analysis methods used are studied in detail in Chapter 6. Examples of the methods are developed, which are then followed by detailed comparisons. The response surface technique has been used in detail to analyse a fixed jacket type structure. An initial screening tool is also presented which can be used to identify the level of complexity of the resistance model required within the system reliability analysis.

The key findings from Chapters 4 to 6 are then collated, and incorporated into an improved and extended version of the framework presented in Chapter 3. The 'final' framework of Chapter 7 presents three levels of detail, and two methods of presentation. The benefits and potential applications of the framework are presented.

The main conclusions focused on a move towards more consistent reliability predictions are described in Chapter 8. Original work and contribution to knowledge are defined, and suggestions made for areas of further work.
CHAPTER 2.

REVIEW STUDY

2.1 Introduction

This chapter 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 and 'true' 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 and 'true' reliability predictions from being made.

This chapter 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.
2.2 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.

2.2.1 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.

### 2.2.2 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.

### 2.2.3 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
- Normal (or Gaussian)
- Weibull
- Exponential
- Poisson
- Log-normal
- Rayleigh

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.

### 2.2.4 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 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 subsystems [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 WSA Atkins 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].

2.2.5 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.
2.2.6 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].

2.2.7 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:

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

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].

2.2.8 Lessons from other industries

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 5).

![Generic decision framework diagram](image)

Figure 5: “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 Birlinshaw, 1992].

2.2.9 Guidelines

There is a definite need within the field of reliability analysis, especially when used in combination with structural integrity analysis, to move towards a set of guidelines in order for a more consistent approach to be adopted. The use of different models, software and users often means variations in methods and assumptions, and this in turn implies that different modelling and statistical uncertainties are included in the analysis.

There is a genuine need to reduce or better quantify modelling uncertainty, as well as to consider alternative means of incorporating modelling uncertainty in reliability analysis. A lack of guidelines or a framework within which such work is undertaken has lead to the development of inconsistent assessments. Other studies [Cornell, 1995; Onoufriou, 1996] have also identified the need for setting guidelines and targets. A framework, in which such aspects are included, combined with information on the interpretation and use of the results is developed within this research. This review study aims to identify all major studies carried out in the past, and to use their combined results to develop such framework and guidelines.

2.3 Uncertainty and sensitivity

2.3.1 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 Muotsu, 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.

2.3.1.1 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."

2.3.1.2 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.

2.3.2 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]:

\[ f'[^0 \mid \text{data}] \propto P(\text{data} \mid ^0) f(0) \]

Where: \( f(0) \) = probability distribution of a parameter, taking a value of 0 before new data,
\( f'(0 \mid \text{data}) \) = probability distribution of 0 afterwards,
\( P(\text{data} | ^0) \) = conditional probability of having observed the new data, were 0 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 $n/n_0$ 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].
2.3.3 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 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:

\[ g(x,\theta) \]

Where: \( g(x,\theta) \) = 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]:

\[ V\{g(x,\theta)\} \]

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

### 2.3.4 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.

2.3.5 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].

2.4 Structural Models

2.4.1 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].

2.4.2 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.

2.4.2.1 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 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.

2.4.2.2 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.

2.4.2.3 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.

2.4.2.4 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</td>
<td>• Fixed piles &amp; rigid joints</td>
<td>• Wave direction</td>
</tr>
<tr>
<td>NW</td>
<td>• Piled foundations &amp; rigid joints</td>
<td>• Foundation stiffness</td>
</tr>
<tr>
<td>SW</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</td>
<td>• Safety critical elements</td>
</tr>
<tr>
<td></td>
<td>with 8 members removed due to low fatigue lives arising from weld defects</td>
<td></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.

2.5 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.

2.5.1 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-Gergersen and Soares, 1997].

2.5.1.1 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; Olufs
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 Skejalbreia 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 \( P(X|X_{mp}) \). The
probability density function of $X_{mp}$ was therefore $p(X_{mp})$. The probability distribution of the extreme value of $X$ in any random storm (r.s.) of unknown $X_{mp}$ is:

$$P(X|\text{r.s.}) = \int p(X|X_{mp})p(X_{mp})dX_{mp}.$$  

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

$$P(X|nT) = [P(X|\text{r.s.})]^nT$$

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:

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

Where: $A$ = crest elevation

$S(\omega)$ = ocean surface energy spectrum

$H_s$ = 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.

### 2.5.1.2 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
\[ u = \text{water particle velocity} \]
\[ C_m = \text{inertia coefficient} \]
\[ a = \text{water particle acceleration} \]

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].
2.5.2 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, WSAtics modelled the current speed as an explicit random variable [Gierlinski et al., 1993].

2.5.3 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:

\[ F = \frac{\rho}{2} \cdot V^2 \cdot C_s \cdot A \]

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
2.5.4 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 0.5 m. \( P(H_s < H_s') \) 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 \( H_{s50} \). 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, \( F_{L \ annual} \), that was represented as:

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

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

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

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

2.5.5 Joint probability of wind, wave and current occurrence

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.

2.5.5.1 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.
2.5.5.2 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]

2.5.5.3 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.

2.5.5.4 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:

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

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>CNS &amp; SNS</td>
<td>0.80</td>
<td>0.84</td>
<td>0.18</td>
<td>0.212</td>
</tr>
<tr>
<td>NNS</td>
<td>0.75</td>
<td>0.81</td>
<td>0.21</td>
<td>0.265</td>
</tr>
<tr>
<td>CNS &amp; SNS</td>
<td>1.00</td>
<td>1.05</td>
<td>0.19</td>
<td>0.180</td>
</tr>
<tr>
<td>NNS</td>
<td>1.00</td>
<td>1.07</td>
<td>0.23</td>
<td>0.215</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>
<thead>
<tr>
<th></th>
<th>Conventional site assessment loads</th>
<th>Shell 100-year return loads</th>
<th>Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Base shear</td>
<td>13.0 MN</td>
<td>6.5 MN</td>
<td>$x 2.0$</td>
</tr>
<tr>
<td>OTM</td>
<td>1083 MNm</td>
<td>604 MNm</td>
<td>$x 1.8$</td>
</tr>
</tbody>
</table>

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]

2.5.5.5 *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:

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

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].

2.5.5.6 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.

2.5.6 Environmental uncertainties and sensitivities

2.5.6.1 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></td>
<td>• Soil p-y curves</td>
<td></td>
</tr>
<tr>
<td>Probabilistic analysis process</td>
<td>• Long term wave, wind &amp; current</td>
<td>• Probabilistic analysis</td>
<td></td>
</tr>
<tr>
<td>leading to Deterministic design process</td>
<td>distributions</td>
<td>• Non linear analysis</td>
<td>• Past experience</td>
</tr>
<tr>
<td></td>
<td>• Joint probabilities</td>
<td>• System behaviour</td>
<td>• Value analysis</td>
</tr>
<tr>
<td></td>
<td>• Parameter uncertainties</td>
<td></td>
<td>• Standard society norms</td>
</tr>
</tbody>
</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 either as 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].

2.5.6.2 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, Hs, 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.
2.5.7 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.

2.6 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.

2.6.1 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.

2.6.1.1 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
- \( \frac{i}{n+1} \) = 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:

\[ \mu_B = -\mu_C \mu_K(x^*) \]
representing random uncertainty

and

\[ \sigma_B^2 = \sigma_C^2 + \sigma_K^2 + (x^*)^2 \sigma_K^2 + 2[x^*] \rho_{CK} \sigma_C \sigma_K \]
representing model uncertainty
Where the statistics C and K are determined using a Bayesian approach and $\sigma_\varepsilon$ 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.

2.6.1.2 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:

$$ P_f = 1 - F_L(Q_a) \equiv \int_{x=Q_a}^{\infty} f_L(x) \, dx $$

Where: $F_L = \text{cumulative probability density function of pile load}$  
$Q_a = \text{actual pile bearing capacity}$  
$f_L = \text{probability density function of pile load}$

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:

$$ Q_s = \int_0^L \pi D \tau_{rz} \, dz $$

Where: $D = \text{diameter of pile}$  
$\tau_{rz} = \text{ultimate skin friction}$
The ultimate skin friction, $\tau_{RZ}$, is defined as follows:

$$\tau_{RZ} = K \cdot \sigma'_{vo} \tan \delta$$

Where:

- $K = \text{stress ratio}$
- $\sigma'_{vo} = \text{initial vertical stresses}$
- $\delta = \text{interface friction angle}$

The stress ratio $K$ is determined as a function of the over-consolidation ratio, OCR, of the clay and the relative density, $R_d$, 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.

### 2.6.1.3 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:

$$\mu_g = \mu_R - \mu_L$$

and

$$\sigma_g = \left(\sigma_R^2 + \sigma_L^2\right)^{0.5}$$

Where:

- $\mu = \text{mean of normal distribution}$
- $\sigma = \text{standard deviation of normal distribution}$
- $g = \text{state function}$
- $R = \text{resistance function}$
- $L = \text{load function}$

The reliability index $\beta$, is defined as the ratio of the mean of the state function divided by its coefficient of variation $V$.

$$\beta = \frac{- \ln \left(\frac{\mu_g}{\mu_R}\right)}{\left(\frac{\sigma_R}{\sigma_L}\right)^{0.25}}$$

The probability of failure is then defined as:

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

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

2.6.2 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:

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

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:

Shaft friction, \( f = K \cdot po' \cdot \tan \delta \)

Where:
- \( K \) = dimensionless coefficient of lateral earth pressure (ratio of horizontal to vertical normal effective stress)
- \( po' \) = 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].

2.6.2.1 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:

\[ \tau_r = \sigma'_{rf} \tan \delta_f \]

Where: \( \tau_r \) = peak local shear stress
\( \sigma'_{rf} \) = radial effective stress at point of shaft failure
\( \tan \delta_f \) = interface angle of friction at failure

Thus the radial effective stress acting on the shaft at failure, \( \sigma'_{rf} \), depends on \( \sigma'_{ri} \), the value acting after installation and full pore pressure and radial stress equalisation, combined with any changes developed during pile loading. The \( \tan \delta_f \) 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 \( \tau_r \) 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:

$$\frac{Q_s(t)}{Q_s(t=\\text{day})} = 1 + A \log(t/t=1 \text{day})$$

Where: $Q_s = \text{shaft capacity}$

$t = \text{time of assessment (up to a maximum of five years)}$

$A = \text{coefficient (value is } 0.5 \pm 0.25)$

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.

<table>
<thead>
<tr>
<th>Method</th>
<th>Mean, $\mu$</th>
<th>Std dev., $\sigma$</th>
<th>COV = $\mu/\sigma$</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 11: Assessment of capacity predictions for piles in sand [Jardine and Chow, 1996b]
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 \( \sigma'_{rc} \), 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>0.98</td>
<td>0.33</td>
<td>0.34</td>
</tr>
<tr>
<td>New method [Jardine and Chow, 1996b]</td>
<td>1.01</td>
<td>0.18</td>
<td>0.18</td>
</tr>
<tr>
<td>Base capacity</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>API RP2A (20th) [API, 1993a]</td>
<td>1.06</td>
<td>1.04</td>
<td>0.98</td>
</tr>
<tr>
<td>New method [Jardine and Chow, 1996b]</td>
<td>0.96</td>
<td>0.18</td>
<td>0.18</td>
</tr>
</tbody>
</table>

Table 12: Assessment of peak clay shaft capacity prediction [Jardine and Chow, 1996b]

2.6.3 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].

2.6.4 **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].

2.7 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].

2.7.1 Different approaches to ultimate capacity prediction

2.7.1.1 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 ($L_{d100}$) 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.

2.7.1.2 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, \( P_{sys,annual} \), that load exceeds the system capacity can be represented by:

\[
P_{sys,annual} = P_{\omega_o} \{ L_{annual} \geq SC \}
\]

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

\[
P_{sys,total} = T_{life} \cdot P_{sys,annual}
\]

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].

### 2.7.1.3 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].

2.7.2 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.

2.7.2.1 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, $B_{fl}$, and axial foundation bias factor, $B_{fn}$. The bias factor was thus:

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

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 $B$:

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

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:

$$l_k (b \mid n \text{- observations}) = \prod_{\text{platform}, i} \left[ l_k (b \mid \text{observation}) \right]$$

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].

2.7.22 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].

2.7.2.3 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.”

2.7.2.4 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.

2.7.2.5 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.

2.7.2.6 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].

2.7.3 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].

2.8 System Effects

System effects in fixed offshore platforms can be divided in to 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].

2.8.1 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.

### 2.8.1.1 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:

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

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.

### 2.8.1.2 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.
2.8.1.3 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]:

\[ RF = \frac{\text{damaged strength}}{\text{strength loss}} = \frac{N_{Lp} - 1}{N_{Lp} - (N_{Lp} - 1)} = N_{Lp} - 1 \]

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]:

\[ RF = \frac{\text{ultimate strength}}{\text{capacity at first member failure}} \]

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 WSAatkins the redundancy factor is calculated for the intact structure from the load factor for global collapse (\(\lambda_{ult}^i\)) and for first member failure (\(\lambda_{ult}^j\)). 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, \(RF = (\lambda_{ult}^i - \lambda_{ult}^j) / \lambda_{ult}^i\), and damage tolerance ratio, \(DTR = \lambda_{ult}^d / \lambda_{ult}^i\) [Gierliński 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.

2.8.2 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.”
2.8.2.1 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:

\[ \text{RSR} = \exp (\beta \sigma - K \sigma_{\ln S}) \]

Where: 
- \( K = \Phi^{-1} (1 - T_s^{-1}) \)
- \( \sigma_{\ln S} = \text{Standard deviation of probability distribution of logarithms of annual expected maximum lateral loadings} \)
- \( T_s = \text{return period in years associated with reference environmental lateral loading} \)
- \( \Phi = \text{standard normal distribution} \)
- \( \sigma = \text{resultant uncertainty in the platform capacity and loading} (\sigma^2 = \sigma_{\ln R}^2 + \sigma_{\ln S}^2) \)
- \( \beta = \text{normalised measure of the structural probability of failure} \)

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: \( P_f = \text{probability of failure during one year} \)
- \( \beta = \text{normalised measure of the structural probability of failure} \)

2.8.2.2 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 WSAAtkins 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 WSAAtkins [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,

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

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

\(\beta_{nfin}\) = 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,

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

Where: \(\beta_{nfin}\) = reliability index for first failure of a member previously identified as critical

\(\beta_{sys}\) = 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: \(\beta_{L1}\) = reliability index derived for the first failure of any member

\(\beta_{nfin}\) = reliability index for first failure of a member previously identified as critical

\(\beta_{sys}\) = 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].

2.8.3 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 [Facciolli et al., 1995] or fracture conditions.

2.9 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.

2.9.1 Indefatigable 49/18AD

2.9.1.1 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:

\[ L = G \left( \xi H^8 + \epsilon C \right) \]

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

\[ L' = X L \]

Where: 
- \( L' \) = posterior load random variable
- \( L \) = original random load variable
- \( X \) = correction value (random variable).

2.9.1.2 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.

2.9.2 Lomond

2.9.2.1 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 the 50-year 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)

2.9.2.2 Results and Conclusions

In the WSA Atkins study, the pushover capacity of the jacket was evaluated under both the design and the extreme environmental loading conditions [WS Atkins, 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” [WS Atkins, 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</td>
<td>RSR=3.38</td>
</tr>
<tr>
<td></td>
<td>Extreme</td>
<td>RSR=2.92</td>
</tr>
<tr>
<td>Damaged</td>
<td>Design</td>
<td>DTR: Case (i) = 0.86, (ii) = 0.90, (iii) = 0.72</td>
</tr>
<tr>
<td></td>
<td>Extreme</td>
<td>DTR: Case (i) = 0.87, (ii) = 0.90, (iii) = 0.74</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)

2.9.3 Leman 49/27 AP

2.9.3.1 Analyses and methodology

In 1997, WSAAtkins completed a study commissioned by Amoco, to perform both deterministic and probabilistic structural analyses of the Leman jacket structure [WSAtkins, 1997b]. Within this study, WSAAtkins 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 as follows:

- deterministic analysis and non-linear response for the 50-year 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.
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

2.9.3.2 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.
Structural System Reliability Framework For Fixed Offshore Platforms

<table>
<thead>
<tr>
<th>Scenario</th>
<th>Ultimate load factor, $\lambda_{ult}$</th>
<th>Deterministic damage tolerance ratio</th>
<th>Annual failure probability (P_{dam})</th>
<th>Reliability index ($\beta_{dam}$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Intact</td>
<td>2.13</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Damage A</td>
<td>1.81</td>
<td>0.85</td>
<td>2.28e-04</td>
<td>3.50</td>
</tr>
<tr>
<td>Damage B</td>
<td>1.71</td>
<td>0.80</td>
<td>4.42e-04</td>
<td>3.33</td>
</tr>
<tr>
<td>Damage C</td>
<td>2.10</td>
<td>0.98</td>
<td>3.80e-05</td>
<td>3.95</td>
</tr>
<tr>
<td>Damage D</td>
<td>1.59</td>
<td>0.75</td>
<td>9.00e-04</td>
<td>3.12</td>
</tr>
<tr>
<td>Damage E</td>
<td>1.41</td>
<td>0.66</td>
<td>2.68e-03</td>
<td>2.78</td>
</tr>
<tr>
<td>Damage F</td>
<td>1.80</td>
<td>0.84</td>
<td>2.45e-04</td>
<td>3.49</td>
</tr>
</tbody>
</table>

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.

2.9.4 Hurricane Andrew JIP

2.9.4.1 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:

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

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:

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

### 2.9.4.2 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.”
2.9.5 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 [Imm et al., 1989]</td>
<td>Reliability index. Bayesian updating</td>
</tr>
</tbody>
</table>

Table 19: Summary of case studies and the effects examined

2.10 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.

2.10.1 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 platform are discussed in the following sections.

2.10.1.1 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, WSAtkins, 1997a].

2.10.1.2 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:

\[ P_{s_y,annual} = \int_0^\infty F_{sc}(x) \cdot f_{L,annual}(x) \, dx \]

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

\( f_{L,annual}(.) \) = 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].

2.10.1.3 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 a review of contributions to offshore technology, a simple expression for the probability of failure was presented thus [Cornell, 1995]:

$$P_f = \int H(x) f_R(x) dx = H(R) e^{\frac{1}{2}(\delta R)^2}$$

Where: $P_f$ = probability of failure

$H$ = complementary cumulative distribution function (CCDF) of the load, $S$

$f_R$ = probability density function of the capacity or resistance

$\delta_R$ = coefficient of variation of the capacity and $R$ = mean capacity

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 into a working approach.

2.10.2 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 20: Summary of probability equations used

<table>
<thead>
<tr>
<th>Method</th>
<th>Probability equation</th>
<th>Definitions</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shell [van de Graaf et al., 1994b]</td>
<td>$P_{\text{g}}(\text{survival}) = P_{\theta} (L_0 &lt; \lambda_0 \cdot S_{\text{ref}})$</td>
<td>$L_0$ = environmental load in direction $\theta$, $\lambda_0$ = collapse load factor in direction $\theta$, $S_{\text{ref}}$ = reference base shear force, $\theta$ = wave attack direction</td>
</tr>
<tr>
<td></td>
<td>$P(\text{survival}) = \Pi_{\theta} P_{\theta} (L_0 &lt; \lambda_0)$</td>
<td>$\Pi_{\theta}$ = product over all wave directions $\theta$ of the severe sector</td>
</tr>
<tr>
<td>DNV/ SINTEF [Sigurdsson et al., 1994]</td>
<td>$P_{\text{f, annual}} = e^{- \int F_{\text{SC}}(x) f_{\text{annual}}(x) dx}$</td>
<td>$F_{\text{SC}}(.)$ = cumulative annual probability distribution of the system capacity, $f_{\text{annual}}(.)$ = probability density function of annual probability distribution of the load</td>
</tr>
<tr>
<td>Cornell [Cornell, 1995]</td>
<td>$P_f = \int H(x) f_R(x) dx \equiv H(R)$</td>
<td>$P_f$ = probability of failure, $H$ = complementary cumulative distribution function (CCDF) of load, $S$, $f_R$ = probability density function of the capacity or resistance, $\delta_R$ = coefficient of variation of capacity, $R$ = mean capacity</td>
</tr>
</tbody>
</table>

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.

#### 2.11 Discussion on review study

The aim of this chapter 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 ‘true’ reliabilities. The chapter 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, and ‘true’ 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.

2.11.1 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

\[ \beta = \text{reliability index.} \]

- 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.
- Reliability results used to be taken as an indication of notional reliability, but more recently, there have been efforts to bring reliability prediction as close to "true" reliability as possible.
- Changes in modelling/software have helped minimise 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. [Onoufriou et al., 1994; Birkinshaw et al., 1994]
2.11.2 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].

2.11.3 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 chapter 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</td>
</tr>
</tbody>
</table>

Table 21: Main issues to be addressed in the development of a generic framework
CHAPTER 3.

INITIAL FRAMEWORK DEVELOPMENT

3.1 Introduction

This chapter introduces a proposal for an initial 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 extensive review presented in the previous chapter. Three different formats have been developed for the initial presentation of the framework. It is concluded that new structures would benefit from an early application of the framework and older structures, undergoing reassessment, would also benefit. The use of the framework would thereby help towards achieving more consistent reliability predictions.

3.1.1 Background

The design requirement of an offshore platform is that it must satisfy the functional need of providing the support structure for offshore oil and gas operations and be structurally adequate for both operational and extreme loading. There are many different loads to be taken into account at the design stage, including dead and live loads, vibration, self weight, ice, ship impacts, wind, wave, tide, current, fatigue, foundation reactions, seismic effects etc.

The framework developed here is specifically designed to assess extreme weather. In the future, other frameworks could address other hazards.

3.1.2 Need for a more rational approach to structural reliability analysis

In the previous chapter it was shown that there is a definite need within the field of reliability analysis, especially when used in combination with structural integrity analysis, to move towards a set of guidelines in order for a more rational approach to be adopted. The development of a generic framework, which will set the basis for achieving more consistent system reliability assessments, was therefore undertaken as part of this research. The main steps involved in a system reliability assessment, together with the key technical and philosophical issues, have been identified and examined. Their inter-relations and relative significance are assessed in order to link them together in a rational process that provides the basis for consistent reliability assessments. The key underlying question throughout this
research is what changes or improvements can be made to reliability assessments in order to move towards more consistent reliability. The perceived benefits of this research include providing a basis for future working practice or guidelines and improved preparation before a reliability analysis is undertaken in order to move towards improved consistency in results.

3.1.3 Identification of technical and philosophical issues

The initial task undertaken in this research was the review study, where the main aim was to identify the "state of the art" in the area of offshore structural reliability. Chapter 2 incorporated an introduction to generic reliability issues and then briefly described all major aspects of reliability analysis. During this review period, the key findings of technical and philosophical issues were identified for incorporation into the subsequent framework. The review study presented in the previous chapter enabled identification of areas where future work was needed to converge towards more consistent reliability predictions. Three main areas for work were identified for further study:

- examination of the effect of certain individual parameters used within the overall reliability analysis process
- scrutiny of the effect of key parts of the process and
- analysis of the different methods of reliability assessment.

These three main elements therefore drove the subsequent framework development phase.

3.2 Initial development of generic framework

The task of presenting all key stages of a reliability assessment, along with the technical and philosophical issues, has never been performed previously. Therefore, this research started from scratch and a generic framework was developed to set the basis for achieving more consistent system reliability assessments. A key underlying question throughout this framework development was what changes or improvements could be made to the reliability assessment process in order to move towards true reliabilities (or failure probability that could begin to be interpreted as absolute values for decision making). This question can be addressed once all the individual steps and issues have been identified, and the areas where uncertainty are introduced have been determined.

3.2.1 Top-level flowchart framework

From the review, the key stages of the assessment process were identified and basic diagrams were drawn up to represent the main steps in the overall process. These rough diagrams were then augmented and developed to form a more detailed approach. Several different options for presentation were explored, with the flowchart type presentation being
the preferred option due to its “visual” impact, its clear presentation of the issues and unambiguous representation of their links. The flowchart is a very concise method of presentation, with only the key characteristics of each step being described. Standard flowchart symbols were used to help the reader to ascertain the status of each step Table 22.

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>□</td>
<td>Process</td>
</tr>
<tr>
<td>□</td>
<td>Input or output</td>
</tr>
<tr>
<td>□</td>
<td>Decision</td>
</tr>
<tr>
<td>□</td>
<td>Document</td>
</tr>
<tr>
<td>□</td>
<td>Terminal (start or end)</td>
</tr>
</tbody>
</table>

Table 22: Standard flow chart symbols used

The framework was then studied further and improved in order to allow a more detailed presentation of the key stages. Tabular formats were developed as an alternative presentation method to the flowchart approach, in order to enable the key background documents at each stage of the framework to be identified, and clearly presented. The use of the tables allowed a full description of the activity to be included, along with an indication of the significance of uncertainty, sensitivity, complexity and level of user competency required at each stage. In order to understand the workings of the generic framework flowchart, a top-level flowchart was drawn up first. This introduces the main elements of the generic framework, without going into the detail of all the steps required within each stage of the process, see Figure 7.

The following section describes each step of the top-level framework. The first symbol used in the top-level generic framework flowchart represents the input, and shows the main inputs required for a reliability assessment. These include details of the platform description, buoyancy effects (if appropriate), details of foundations and soil condition, and details of the environmental conditions to be applied. The second stage is represented by the standard symbol for a process and relates to the assessment of the fixing of the structure. This means an appraisal of the foundation conditions and the foundation configuration.

The third stage process is the undertaking of the modelling of the structure. Decisions concerning structural members, nodes and elements, along with the parts to be modelled (e.g. decks and equipment) are made here. The result is a sufficiently detailed description of the structure that meets the precise needs of the study. Decisions relating to what software package is to be used will also be made at this stage.
The fourth stage, again a process, represents the derivation of the foundation capacity and its stiffness from the foundation capacity and distribution, structural configuration and the soil characteristics specific to the precise location of the structure. This is performed in combination with the derivation of the loads on the structure. This is based on an assessment of the statistical distribution of the environmental parameters anticipated to be acting upon the structure.

The fifth stage is the process of derivation of the system analysis model, and involves complete structural analysis using various software options, platform model, loads etc. The sixth stage is the process of derivation of the ultimate capacity of the structure. The process undertaken in this activity will depend on whether a "component" based approach is adopted or whether a "system" analysis approach is used. The seventh stage of the flowchart is the process of undertaking a reliability analysis, and the determination of the associated uncertainty, using the results of previous stages.

Figure 7: Top-level generic framework flowchart

The eighth and final stage of the flowchart is the output of the whole procedure. It is the determination of the probability of failure of the structure from a study of the failure surface in combination with the uncertainty descriptions derived at the seventh stage. A determination of the reliability of the structure will be derived from the probability of failure and the uncertainty analysis.
3.2.2 Detailed generic flowchart framework

Once the top-level generic framework had been completed, the next task performed was to augment each of the stages. This involved the identification of each of the separate tasks necessary to perform that particular stage of the analysis. The result was a detailed generic framework, which shows each step that needs to be carried out and the precise sequence for those activities, in order for a full structural system reliability assessment to be undertaken. The flowchart includes all the activities necessary to collate the inputs required, to perform the processes required and to produce all the outputs required.

The sequence and contents of each step of the structural reliability analysis were based on information obtained during the literature survey. Initially, the detailed framework was developed to include all aspects of assessment for any general type of offshore structure (fixed jacket, gravity based, semi-submersible etc). The framework also incorporated all aspects for the reliability assessment of both design and reassessment of offshore structures.

Despite the fact that the framework was developed as a flowchart to exploit its “visual” impact and clear presentation of the issues and unambiguous representation of their links, it is a complex and intricate flowchart. This means that it needs to be studied carefully and in detail in order to fully appreciate all the steps required for a structural reliability assessment.

The influence of the user has been identified at a number of stages in the reliability assessment process. It is summarised in one input box, which is described on the framework as “assumptions, judgement and knowledge of user”. This is used to incorporate all aspects of user influence on any given task. However, human error is not represented at any of the stages, as it was considered out of the scope of the current research.

Areas where uncertainty was thought to be included in a reliability analysis were identified by a documentary output symbol, which recommended that the user should “determine associated uncertainty”. In certain circumstances, it may be very difficult or impossible to derive an actual value for the uncertainty, but this step is specifically designed to bring this fact to the user’s attention.

There are only two places on the whole framework where a decision step is immediately followed by two alternative approaches. One such branching occurs after the assessment of structure fixing conditions, which is followed by one route for fixed structures and another route specifically for floater structures. The location of the other branching is immediately after the decision concerning whether the structure requires foundation assessment or not. This is basically along similar lines to whether the structure is fixed or floating.
Figure 8: Generic framework for both new (design) and old (reassessment) structures
The generic framework has been split into three parts for ease of presentation: the first part shows the technical and philosophical issues and their relationships for the initial activities required. Options are included in this part of the generic framework for both new/design and old/reassessment structures - inspection reports, weld defect assessments and specific damage or defect reports are included for the old/reassessment structures in order to provided a condition assessment of the overall structure. Options are also included for fixed and floater structures - for the floater structures, an assessment of the buoyancy effects and their distribution, along with the associated uncertainty, is required. The determination of structural members, inclusion of structural parts and decisions as to how these are to be modelled are included in this part.

Figure 9: Generic framework for design and reassessment of structures - part 1
The second part of the generic framework involves the assessment of the local environmental conditions and the foundation assessment. Determination of the environmental parameters statistical distribution and derivation of loads on the structure, combined with the determination of the associated uncertainty are performed at this stage. Options are presented in the foundation assessment according to whether or not a foundation assessment is actually required - if it is required, then an assessment of the conditions from the local soil conditions, the influence of ageing and group interactions, along with the uncertainty.

Figure 10: Generic framework for design and reassessment of structures - part 2

\[= \text{Process}, \bigtriangleup = \text{Input / output}, \triangleleft = \text{Decision}, \bigcirc = \text{Document}, \square = \text{Terminal (start/end)}\]

The third and fourth parts of the generic framework involve a decision as to which reliability approach to adopt, and then follow the procedure required for each option. The methods covered are both the “component” based approach and the “system” analysis approach. Within the “component” based approach there are three techniques which can be applied -
“minimal” analyses approach (e.g. where only 8 analyses are carried out - one for each wave direction), response surface technique, and the numerical simulation approach. The performance of non-linear pushover analyses and decisions on failure criteria, determination of the ultimate capacity and distribution of strength and hence the probability of failure, along with integration of strength with loading are all included. Within the “system” analysis approach, the dominant failure modes and reliability analysis, together with calculation of reliability and sensitivity measures of the system, are required. For both approaches, presentation of associated uncertainties is also required, before the final stage of deriving the measure of reliability is achieved. The final activity is comparing reliability with pre-defined targets and acceptance criteria.

Figure 11: Generic framework for design and reassessment of structures - part 3
3.2.3 Tabular framework

As described earlier, the framework was developed in three distinct forms: generic overview, flowchart and tabular formats. The tabular format allows a much greater depth of detail to be presented and allows the user to home in on a specific stage of the reliability assessment process if required. It has also been adapted to include the main references pertaining to each of the activities to enable the framework to be traceable, and to simplify further study of certain aspects. Table 23 shows the outline table which describes the basic stages including main inputs and outputs.
**INPUTS**
- Description of platform structure (from design drawings, defect/damage/condition reports, computer models etc.)
- Deadload and liveload parameter values (applicable to "floater" structures: incl. buoyancy effects, inertial/dynamic parameters)
- Foundation parameter values (from soil conditions, pile conditions, group interaction etc.)
- Environmental parameter values (wave height & period, current, wind, inertia, drag etc.)

**Stage 1. ASSESSMENT OF FIXING OF STRUCTURE**
- Assessment of fixing conditions of structure (to determine whether fixed or "floater")
- For "floater" structures: determination of modelling method for buoyancy effects

**Stage 2. MODELLING OF STRUCTURE**
- Decision as to what software package to use (may be governed by external constraints)
- Determination of structural members (i.e. members/parts to model and in what detail)

**Stage 3. CAPACITY AND LOAD DERIVATION**
- Determination of foundation capacity and stiffness (from capacity and distribution etc.)
- Determination of environmental loads on the structure (from environmental parameters distribution, statistical distribution etc.)

**Stage 4. SYSTEM ANALYSIS MODEL DERIVATION**
- Complete structural analysis using various software options (platform model, loads etc.)

**Stage 5. ULTIMATE CAPACITY DERIVATION**
- Decision as to which reliability methodology to adopt: either "component" or "system" based (may be governed by external constraints)
  - For "component" based approach:
    ⇒ Perform pushover analysis to identify dominant failure modes
    ⇒ Perform either: Minimal analyses/ response surface/ numerical simulation approach
    ⇒ Perform number of pushover analyses (determine load-deformation characteristics)
    ⇒ Decide on failure criteria e.g. determine ultimate capacity & other failure characteristics, & failure surface if required (from pushover analyses results)
    ⇒ Determination of distribution of strength (dependent upon the focus of the study)
    ⇒ Integrate distribution of strength with loading (e.g. extreme envt loading)
    ⇒ Present assessment of uncertainties to determine uncertainty in strength (on both member and system level, if required)
  - For "system" analysis approach:
    ⇒ Find dominant failure modes from search algorithms (decide failure surface if reqd)
    ⇒ Build up structural system, including series of parallel sub-systems if required (dependent upon the focus of the study)
    ⇒ Perform "component" reliability analysis, and determine associated uncertainty
    ⇒ Calculate reliability of system, and sensitivity measures
- Present, understand and interpret results

**OUTPUT**
- Determination of probability of failure
  (from study of failure surface, in combination with uncertainty descriptions)
- Determination of measure of reliability of structure
  (from probability of failure and uncertainty analysis)

Table 23: Summary outline generic framework presented in tabular format
Each stage identified in the outline generic framework table has been examined further, and a detailed breakdown of every single activity required has been produced. A sample is given in Table 24 relating to Stage 2: modelling of structure.

<table>
<thead>
<tr>
<th>Stage 2. Modelling of structure</th>
<th>Brief description of activity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Input 1</td>
<td>From Stage 1: description of platform structure &amp; fixing conditions (from design drawings, computer models etc.)</td>
</tr>
<tr>
<td>Step 2.1</td>
<td>Decision as to what software package to use (may be governed by external constraints)</td>
</tr>
<tr>
<td>Step 2.2</td>
<td>Determination of structural members</td>
</tr>
<tr>
<td>Step 2.3</td>
<td>Assessment of relevance or importance of structural parts (e.g. is a detailed deck necessary?)</td>
</tr>
<tr>
<td>Step 2.4</td>
<td>Decision as to what structural parts are to be included in the model</td>
</tr>
<tr>
<td>Step 2.5</td>
<td>Presentation of valid reasons why certain parts are not included</td>
</tr>
<tr>
<td>Step 2.6</td>
<td>User discretion &amp; interpretation of the environmental loads (User effects are assumptions, judgment &amp; knowledge)</td>
</tr>
<tr>
<td>Step 2.7</td>
<td>Decision as to how parts are to be modelled</td>
</tr>
<tr>
<td>Step 2.8</td>
<td>Presentation of the justification for the modelling method chosen</td>
</tr>
<tr>
<td>Output 1</td>
<td>Full model of structure appropriate to &amp; specific to the current assessment being undertaken</td>
</tr>
</tbody>
</table>

Table 24: Detailed breakdown table for Stage 2: Modelling of structure

3.3 Framework specific to the design of fixed offshore platforms

In order to examine future potential developments of the generic framework, a framework for a specific application has been derived. This framework was developed to be suitable for use in the design of fixed offshore structures in the North Sea.

Figure 13 shows the specific framework for the design of fixed offshore platforms as a flowchart. This flowchart was based on the generic framework, but has been changed and improved to reflect only those issues which relate to the design stage and not the reassessment stage of a structure. The flowchart also only deals with fixed jacket type structures and does not deal with the issues relating to floater structures.
Deterministic distribution of strength (member / structure) & obtain probability of failure

Assess the reliability of designer's work

Comparison reliability with predetermined targets & acceptable criteria

Figure 13: Specific framework for design of fixed offshore platforms

- Process,  = Input / output,  = Decision,  = Document,  = Terminal (start/end)
The generic outline table that described the basic stages for the whole process, including the main inputs and main outputs, was also the basis for the detailed example exercise. The stages identified in this table were examined in turn and a full table of each step to be performed at each stage was developed in more detail. These detailed tables were also developed to incorporate indication of the following for each step to be performed:

- uncertainty (U) - amount of uncertainty introduced at each step
- sensitivity (S) - sensitivity of overall reliability results to each step
- complexity (C) - level of complexity of the actions for each step
- operator competence (O) - user competency required, perceived importance.

This star scale has been adopted for the four factors, based on a qualitative assessment and engineering judgement. One star indicates "low", five stars indicate "high". This scale is an attempt to qualitatively indicate levels, but it should not be interpreted as a definitive representation. At a generic level, all four of the above factors are considered to be of importance. However, during the examination of the specific example, it was found that the complexity and user competence factors were often awarded the same level. It was considered important that this should be identified, and even though having both factors may not be fully justified in this specific case of the design of fixed offshore platforms in the North Sea, it may become more pertinent for different specific examples.

<table>
<thead>
<tr>
<th>Step</th>
<th>Brief description of activity</th>
<th>Arbitrary scales</th>
</tr>
</thead>
<tbody>
<tr>
<td>Input 1</td>
<td>From Stage 1: description of platform structure and fixing conditions (from design drawings, computer models)</td>
<td>** *** ** **</td>
</tr>
<tr>
<td>Step 2.1</td>
<td>Decision as to what software package to use (may be governed by external constraints)</td>
<td>** ** **</td>
</tr>
<tr>
<td>Step 2.2</td>
<td>Determination of structural members</td>
<td>**** **** *** ****</td>
</tr>
<tr>
<td>Step 2.3</td>
<td>Assessment of relevance/importance of structural parts (e.g. is a detailed deck necessary?)</td>
<td>***** ***** **** ****</td>
</tr>
<tr>
<td>Step 2.4</td>
<td>Decision as to what structural parts are to be included in the model</td>
<td>**** **** *** ****</td>
</tr>
<tr>
<td>Step 2.5</td>
<td>Presentation of valid reasons why certain parts are not included</td>
<td>**** **** *** ****</td>
</tr>
<tr>
<td>Step 2.6</td>
<td>User discretion and interpretation of the environmental loads (User effects are assumptions, judgment and knowledge)</td>
<td>***** ***** **** ****</td>
</tr>
<tr>
<td>Step 2.7</td>
<td>Decision as to how parts are to be modeled</td>
<td>**** **** *** ****</td>
</tr>
<tr>
<td>Step 2.8</td>
<td>Presentation of the justification for the modelling method chosen</td>
<td>**** **** *** ****</td>
</tr>
<tr>
<td>Output 1</td>
<td>Full model of structure appropriate to and specific to the current assessment being undertaken</td>
<td>***** ***** **** ****</td>
</tr>
</tbody>
</table>

Table 25: Framework Stage 2. Modelling of structure
### Structural System Reliability Framework For Fixed Offshore Platforms

#### INPUTS
- Description of platform structure  
  (from design drawings, defect/damage/condition reports, computer models etc.)
- Foundation parameter values  
  (from soil conditions, pile conditions, group interaction etc.)
- Environmental parameter values  
  (from wave height, wave period, current, wind, inertia, drag etc.)

#### Stage 1. MODELLING OF STRUCTURE
- Decision as to what software package to use (may be governed by external constraints)
- Determination of structural members (i.e. which members/parts to model, in what detail)

#### Stage 2. FOUNDATION CAPACITY AND LOAD DERIVATION
- Determination of foundation capacity and stiffness  
  (from capacity and its distribution etc.)
- Determination of environmental loads on the structure  
  (from environmental parameters distribution, statistical distribution etc.)

#### Stage 3. SYSTEM ANALYSIS MODEL DERIVATION
- Complete structural analysis using various software options (from platform model, loads etc.)

#### Stage 4. ULTIMATE CAPACITY DERIVATION
- Decision as to which reliability methodology to adopt: either “component” or “system” based (may be governed by external constraints)
- For “component!” based approach:
  - Perform pushover analysis to identify dominant failure modes
  - Perform either: minimal analyses, response surface or numerical simulation approaches
  - Perform number of pushover analyses (determine load-deformation characteristics)
  - Decide on failure criteria e.g. determine ultimate capacity & other failure characteristics, & failure surface if required (from pushover analyses results)
  - Determine of distribution of strength (dependent upon the focus of study)
  - Integrate distribution of strength with loading on structure (extreme envt loading)
  - Present assessment of uncertainties to determine uncertainty in strength (on both member and system level, if required)
- For “system” analysis approach:
  - Identify dominant failure modes (decide on failure surface)
  - Build up structural system, including series of parallel sub-systems if required (dependent upon the focus of the study)
  - Perform “component” reliability analysis, and determine associated uncertainty
  - Calculate reliability of system and sensitivity measures
  - Present, understand and interpret results

#### OUTPUT
- Determination of probability of failure  
  (from study of failure surface, in combination with uncertainty descriptions)
- Determination of measure of reliability of structure  
  (from probability of failure and uncertainty analysis)

---

Table 26: Summary outline framework specific to design of fixed offshore platforms
3.4 Different reliability assessment methods

Due to the complexity of the final stages of the framework that deal with the different options available for performing reliability assessments, the following section deals with this in detail and provides examples of the different approaches. This part of the generic framework involves a decision as to which reliability approach to adopt and then follows with the procedure required for each option. The methods covered are both the “component” based approach and the “system” analysis approach. The term “component” here refers to methods that treat the whole structure as one component. Within the “component” based approach there are three techniques that can be applied: minimal analyses approach, response surface technique and numerical simulation approach.

This part of the framework includes the performance of non-linear pushover analyses and decisions on failure criteria, determination of the ultimate capacity and distribution of strength and hence the probability of failure, along with integration of strength with loading. Within the “system” analysis approach, the dominant failure modes and reliability analysis, together with calculation of reliability and sensitivity measures of the system, are required. For both approaches, presentation of associated uncertainties is also required before the final stage of deriving the measure of reliability is achieved. Figure 14 shows the extract of the framework detailing options and steps of the reliability assessment stage.

![Figure 14: Generic framework - reliability assessment extract](image-url)
3.4.1 Minimal analysis approach

The ‘minimal’ analysis option is shown in the Figure 15 where pushover analysis is used to determine the dominant failure modes for each direction and develop failure surfaces. An example of this analysis method is the Shell approach [Tromans et al., 1993; van de Graaf et al., 1993; van de Graaf et al., 1994a; van de Graaf et al., 1994b; Si Boom et al., 1993; Vanderschuren et al., 1996; Vugts and Edwards, 1992; Efthymiou et al., 1997]. Shell refined the deterministic pushover approach in the mid-1990s that forms an important element of the methodology they developed for evaluating the reliability of a platform. Shell’s essential elements to a quantitative reliability analysis are a hindcast database of waves, currents and winds, a realistic wave load model, a generation of extreme long term statistics and the use of pushover analyses.

Figure 15: Framework extract showing steps involved in the “minimal” analysis approach

The major phases of this process are described below [van de Graaf et al., 1994b]:

- A hindcast database of metocean conditions (magnitudes and directions of winds, waves and currents generated numerically) is used to produce a
representative combination of extreme environmental conditions required to generate a reference load set.

- The reference load set is applied to a FE model and increased in increments to obtain the collapse sequence of the structure and its ultimate resistance.
- The reference load is applied several directions to develop a failure surface for the structure.
- Return to using the hindcast data and study the extreme response of a similar "generic" structure in order to calculate the probability that the loading will fall outside the failure surface.
- The probability of failure is obtained by convolution of the cumulative directional distribution of long-term extreme loads with the distribution of the ultimate capacity of the structure.

The probability of survival, $P_0$, under extreme loading in a narrow sector centred on the wave attack direction $\theta$ is defined as follows:

$$P_0(\text{survival}) = P_0 (L_\theta < \lambda_\theta \cdot S_{\text{ref}})$$

Where: $L_\theta$ = environmental load in direction $\theta$, $\lambda_\theta$ = collapse load factor in direction $\theta$ and

$S_{\text{ref}}$ = reference base shear force.

Assuming independence of rare events the probability of long-term survival under extreme loading is the product of probabilities of surviving the extreme loading predicted for each separate wave attack direction. It is given as follows:

$$P(\text{survival}) = \Pi_{\text{all}0} P_0 (L_\theta < \lambda_\theta)$$

Where: $\Pi_{\text{all}0}$ = product over all wave directions $\theta$ of the severe sector.

### 3.4.2 Response surface technique

The second option within the "component" based approach is the use of the response surface technique as shown in Figure 16. This method generates 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 resistance basic 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. adopted the response surface technique in a comparison study of the reliability of fixed offshore structures to the reliability of jack-ups [Frieze et al., 1999; MSL Engineering, 1997].

3.4.3 Numerical simulation approach

One method of deriving the most dominant failure paths is to perform a pushover analysis. The most critical elements are identified in this analysis, but no account is taken of the effect of possible variations in component strength that could result in different sequences of failure involving different combinations of elements. The effects of these variations can be explored more fully using simulation techniques. This is the third option identified in this part of the framework, as shown in Figure 17. However, its use is limited in the case of offshore platforms because of the large scale of the problem.

One of the key studies involving numerical simulations techniques was a study by DNV/SINTEF in 1994 [Sigurdsson et al., 1994]. The DNV/SINTEF approach used the
Structural System Reliability Framework For Fixed Offshore Platforms

programme USFOS for non-linear structural collapse analysis and PROBAN for the probabilistic analysis tool. PROBAN was used to perform the reliability calculations and to generate the outcome of the stochastic parameters in the simulation studies of the ultimate capacity of a structure [Sigurdsson et al., 1994].

The procedure adopted using USFOS and PROBAN can be summarised as follows [Sigurdsson et al., 1994]:

1. Establish wave/current load pattern for a given position of a fixed wave, e.g. the 100 year wave height and the 10 year current pattern.
2. Choose the number of realisations, NSIM, (here NSIM=100)
3. Sample NSIM sets of outcomes of the stochastic variables, i.e. \( Y_i \) (yield strength), \( \epsilon_i \) (imperfection magnitude) and \( \theta_i \) (imperfection length) for each structural member \( i \), using PROBAN.
4. For each set of outcomes of the stochastic variables:
   - Perform a static pushover analysis by scaling up the load profile, as established in step 1 above.
   - Save the results (e.g. displacement of the deck vs. the total base-shear force)

Figure 17: Framework extract showing steps involved in the numerical simulation approach

The procedure adopted using USFOS and PROBAN can be summarised as follows [Sigurdsson et al., 1994]:

1. Establish wave/current load pattern for a given position of a fixed wave, e.g. the 100 year wave height and the 10 year current pattern.
2. Choose the number of realisations, NSIM, (here NSIM=100)
3. Sample NSIM sets of outcomes of the stochastic variables, i.e. \( Y_i \) (yield strength), \( \epsilon_i \) (imperfection magnitude) and \( \theta_i \) (imperfection length) for each structural member \( i \), using PROBAN.
4. For each set of outcomes of the stochastic variables:
   - Perform a static pushover analysis by scaling up the load profile, as established in step 1 above.
   - Save the results (e.g. displacement of the deck vs. the total base-shear force)
The annual system failure probability was determined as the annual probability that the load exceeds the system capacity. This can be defined as:

\[ P_{sys,annual} = \int_{0}^{\infty} F_{sc}(x) \cdot f_{load,annual}(x) \cdot dx \]

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

\( f_{load,annual}(\cdot) \) = 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 \( Z_{sea-state} \), (a vector of random variables modelling the uncertainties in the sea-state description) especially the significant wave height.

3.4.4 System analysis approach

Rigorous “system” reliability analysis requires substantial computations, and research work in recent years has concentrated on the development of efficient methods for identifying the most dominant failure paths and deriving the combined system probability of failure. An example of this approach is that used by WSAtkins. A number of other simplified “system” reliability approaches have also been developed and examples are briefly outlined in this section. Figure 18 shows the framework for the “system” analysis approach.

WSAtkins use system reliability analyses to identify dominant failure modes and to calculate system reliability measures. 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 the basic variables. All important environmental parameters are modelled as explicit random variables.

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, WSAtkins, 1997a].

Figure 18: Framework extract showing steps involved in the “system” analysis approach

There are also a number of simplified “system” reliability methods - these are less commonly applied but are discussed briefly in the sections below. 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 et al [Bea et al., 1997], was applied to the reassessment and re-qualification of two Gulf of Mexico platforms. 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 the USA using the program ULSEA (Ultimate Limit State Equilibrium Analysis) developed in 1995.
In a recent review of contributions to offshore technology, Cornell [Cornell, 1995] presented a simple expression for the probability of failure:

\[ P_f = \int H(x) f_R(x) dx = H(R) e^{\frac{\delta_R^2}{2}} \]

Where: 
- \( P_f \) = probability of failure
- \( \delta_R \) = coefficient of variation of the capacity
- \( H = \) complementary cumulative distribution function (CCDF) of the load, \( S \)
- \( f_R = \) probability density function of the capacity or resistance
- \( R = \) mean capacity.

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” [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 1997, Advanced Mechanics and Engineering, AME developed a new method for assessing the strategic level of structural safety. A need was identified to have a form of modelling which could accommodate the aspects of structural safety associated 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 [Walker, 1997]. 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 into a working approach.

3.5 Discussion of the initial framework

The aim of this chapter was to describe the initial development of a generic framework for system reliability assessment, concentrating on the main steps in a reliability assessment and the key related technical and philosophical issues. These issues have been linked together in a flowchart arrangement, in an attempt to present a rational and concise framework. Three main areas for work were focused on:

- examination of the effect of certain individual parameters used within the overall reliability analysis process
• scrutiny of the effect of key parts of the process and
• analysis of the different methods of reliability assessment.

A generic framework has been prepared which has been developed for use for both design/new and existing/reassessment structures and is applicable to both fixed and floater types of installations. The flowchart presentation format allows, at a glance, the relations between all the issues raised to be presented. It is a concise and succinct method of presentation, although it does not allow in-depth detail of the steps to be presented. It was for this reason that the outline tabular framework was also developed.

In order to study the generic framework in more detail and to break down each stage into individual step activities, a specific example had to be adopted before a more detailed framework could be presented. This is because different issues are raised for different types of structure and it was unwieldy to try to include all structural options within one detailed framework. It was for this reason that the specific example of the design of fixed offshore platform within the North Sea was adopted and developed further. A similar development to a more detailed form can be undertaken for different offshore applications.

The specific example framework that was developed has been presented as both a flowchart and as detailed tables. This latter method allows for much more detail of each step, as well as an indication of the uncertainty, sensitivity, complexity and level of user competency required at any given stage. The relative significance of these factors can be seen at a glance through the arbitrary scale system adopted. This discretionary scale system could be taken further, if required, by sorting and ranking the steps of the framework according to any one of the factors identified. All four factors may not be required for every detailed application since, although separate, the issues of complexity and user competence are undoubtedly linked.

The framework developed has included all the main steps identified through the review study. A top-level framework indicates the main elements of the generic framework starting with the inputs required relating to platform description, foundation and environmental parameters. The next eight stages deal with the detailed steps required to undertake a reliability assessment and include an assessment of the fixing of the structure, modelling of the structure, foundation capacity derivation, load derivation, system analysis model derivation, ultimate capacity derivation and then the reliability analysis. The final stage is the collation of outputs of each of the stages, resulting in a measure of reliability and comparisons with reliability targets if required.
The key underlying question throughout the framework development was what changes or improvements could be made to the system reliability assessment process in order to improve consistency and move towards "true" reliability (or failure probability that could begin to be interpreted as absolute values for decision making)? The framework developed in this chapter provides a sound basis for more consistent application of the reliability techniques. Furthermore, it summarises the whole assessment process so that individual steps can be identified, studied and improved, thus leading to an improvement in overall reliability assessments.

A study of the generic framework has indicated those areas where external constraints are likely to impinge on the activities. These are mainly shown to affect the choice of software and methodology, and the provision of different types of data. There may be constraints from the nature or amount of specific data available. Any constraint may affect the inputs and hence performance of the overall reliability procedure. Although these constraints cannot easily be altered, it was felt important that those steps where constraints would be incurred, should be identified.

The following are examples of the areas identified from the generic framework where external constraints are likely to impinge:

- Stage 2. Modelling of structure: Decision as to what software package to use
- Stage 4. System analysis model derivation: Complete structural analysis using various software options
- Stage 4. Ultimate capacity derivation: Decision as to which methodology to adopt.

### 3.6 Identification of areas of focus for more detailed studies

#### 3.6.1 The effect of individual parameters

The generic framework was studied with a view to focusing the offshore study as the next phase of the research. This enabled identification of areas where future work is needed to improve the methods to converge towards true reliability predictions. These areas were primarily where significant uncertainty is currently incurred or where parameters or processes appear highly sensitive, and hence where further work should be focused. These general areas are as follows:

- Determination of environmental parameters' statistical distributions
- Determination of foundation capacity and its distribution
- Decision on modelling method of foundations
- Performance of pushover analysis and determination of ultimate capacity
• Determination of strength distribution (member or structure)

• Derivation of probability of failure.

Further examination of the general areas identified above, taking into account where previous research has been focused, indicates that the areas of most significance to reliability assessments within the offshore industry are determination of foundation capacity and its distribution and decision on modelling method of foundations.

More specifically, the issue of parameter sensitivity could be studied and in particular the following aspects were identified from preliminary examination of the framework: foundation parameters, modelling of wave load, wave height, drag coefficient, yield strength and imperfection magnitude and direction.

When the foundations are ignored, then the yield strength has previously been found to be a dominant factor on the resistance side. However, no investigations to date have looked into the effects of foundation capacity and stiffness, alone or in conjunction with investigations into the effect of changing the yield strength. Therefore, these two issues will form the basis of the parametric studies (see Chapter 4).

3.6.2 The effect of key parts of the process

During the framework development phase, it has been found that there are significant differences that exist in the way in which the foundation capacity is assessed. This has a significant role in the prediction of the resistance of the structure in order to perform a structural reliability assessment. Further work in this area will therefore address this issue in more detail (see Chapter 5).

3.6.3 Analysis of different methods of reliability assessment

The uncertainty of individual steps in the reliability analysis procedure was also an area identified for further work, including the specific approach or method adopted, modelling uncertainty, study of foundation effects and study of system effects. The variability in different methods such as search algorithms, probability criteria, pushover analysis assisted by simulations and simplified system reliability methods has also been studied in the offshore study in the next phase of the research. The different method adopted for structural reliability analysis will be examined in more detail (see Chapter 6).
CHAPTER 4.

SENSITIVITY STUDY OF INDIVIDUAL PARAMETERS

4.1 Introduction

4.1.1 Background

The generic framework presented and studied in the previous chapter enabled identification of areas where future work was needed to converge towards more consistent reliability predictions. Three main areas for work were identified for further study:

- examination of the effect of certain individual parameters used within the overall reliability analysis process
- scrutiny of the effect of key parts of the process and
- analysis of the different methods of reliability assessment.

4.1.2 Scope of work

This chapter addresses the first area for further work and is centred on assessing the sensitivity of structural reliability to individual parameters. A study of the sensitivities of given variables can be used to 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, it is important to model the variable by using the best available data and distribution.

As discussed in Chapter 2, significant work has been performed to date on parameters that are incorporated into the loading model. Some aspects of the resistance model, however, have not been investigated sufficiently and it is here that this part of the research is currently focused. From the literature studied, it was concluded that the parameter of most significance to response modelling within reliability assessments was the yield strength of the structural material. This is relevant when considering a failure scenario dominated by jacket failure where possible foundation failure has been ignored. If, however, foundations are included in the analysis model, the effect of the capacity and stiffness of the foundations...
of the structure has been identified as an area for more detailed study. This chapter therefore focuses on the combined examination of the yield strength, which has previously been identified as a key resistance parameter for jackets, and the foundation capacity, which has not been previously examined in any great detail. The two key parameters identified here, were studied in a deterministic manner using structural analysis techniques, in order to assess the sensitivity of the structural system response to any changes. This is then followed by a study of the interaction of these two key parameters and their effects on system strength for different failure modes.

It should also be noted that modelling uncertainty was also identified as an important issue to be addressed in order to move towards more consistent reliability assessments. However, this issue has not been addressed in detail in this investigation, but it is an area where further work could usefully be focused.

4.2 Procedure adopted for structural analysis assessments

4.2.1 Introduction

The role of structural analysis is to predict the performance of a structure and is often based on the analysis of mathematical models. The accuracy of the prediction depends on how well the model characterises the behaviour of the structure. Consequently, it is important to know the limitations of the mathematical models used to represent the structure and to realise that the model is only as good as the assumptions and knowledge used to build it.

A model of a structure can be defined as a mathematical representation of the behaviour of the structure in its environment. It is expressed as an action-response relation. Actions are mathematical models of such environmental factors as loads. The response is a measure of the change in state of the structure and is commonly expressed in terms of displacements, strains, stresses and forces. In essence, structural analysis is concerned with the specification of actions, the construction of model(s) of the structure, and the determination of the response of imposed actions. This can be simply represented in the block diagram in Figure 19:

![Figure 19: Simplified representation of the structural analysis process [Holzer, 1985]](image)

The basis of the mathematical model and the procedure adopted for its analysis will be prone to some degree of uncertainty or error and therefore can be used to represent an...
'informed judgement' of how the structure will behave. It is important that detailed calibration exercises are undertaken to establish that the mathematical model is robust enough to encapsulate the full range of structural characteristics. Ideally, this would be carried out with reference to a full-scale model but, as is often the case, this is prohibitively expensive and small-scale simplified models are often used to form the basis for the calibration. Uncertainty may also be introduced in the assumption that the structure is built precisely as designed, which may not always be the case. Slight modifications to the design may be made during the construction phase of a structure, which are not necessarily included in the original design. However, although such an issue is recognised, its consideration is beyond the bounds of this current research.

The approach used in this research was a 3D non-linear finite element analysis technique. The software package used in this research was SAFJAC (Strength Analysis of Frames and JACkets) [BOMEL, 1992]. For this project it was mounted on a Sun Ultra 60 Workstation and operated from UNIX. The program is intensely 'resource hungry' and requires substantial computing power in order to process the analysis. Analysis times varied from 2 to 4 days. The model used corresponded to the offshore platform Leman AP, which consisted of 698 structural nodes and 931 elements, as shown in Figure 20.

Figure 20: SAFJAC model of Leman AP platform, showing location of nodes
As can be seen, Leman AP is six legged. The notation adopted for the location of the piles is shown diagrammatically in Figure 21:

\[ \text{Rows 1 2} \]

Figure 21: Diagram showing pile layout for Leman AP platform with true and platform North

SAFJAC was developed for the non-linear analysis of frames, with particular emphasis on the estimation of reserve strength of tubular framed structures. It has advantages over the more general, conventional finite element packages such as ABAQUS [Hibbitt et al., 1995]. In particular, the basic beam-column elements are modelled in SAFJAC using a quartic formulation, which enables each member to be modelled as one element. This is only possible due to the fact that the formulations were specifically developed to accurately encompass large deflection behaviour [BOMEL, 1997].

SAFJAC also incorporates the use of adaptive finite element techniques – this is where the mesh is refined and the number of nodes and elements are increased as the solution progresses. New elements and nodes are thus created during the analysis as the material yields and behaves non-linearly - such a phenomenon occurs whenever plasticity is detected within a member. This means that the model size can be kept to a minimum, as only the areas where the mesh needs to be finer are modelled in more detail.

4.2.2 Loading application

The loading was applied to the structural model in two stages: initial loading and proportional loading. The first stage was the initial loading that corresponded to the still water condition including dead loads and also incorporated wind loading on the topside at this stage. The second stage was the proportional loading that corresponded to the environmental loading from the 50-year extreme storm wave and current. Proportional loads were increased to determine the ultimate load of the structure. All loads were applied as concentrated nodal forces. The wave loading was primarily applied to vertical members exposed to the wave approach direction, with the highest nodal forces being at the bay at an elevation of +39.46m. This is located at the top of the x-braced jacket structure below the cellar deck and is in the region where the 50-year still water height is located.
4.2.3 Structural idealisation

4.2.3.1 Modelling

The fundamental beam element used within SAFJAC is an elastic high-order quartic beam-column element. High-order is a term used here as an indication that a large number of points (nodes) and surfaces (faces) define the element. Non-linear spring elements are used to represent joint flexibility and pile-soil interaction. The spring behaviour in pile tension and compression may be different and a feature allows this to be modelled independently.

A plastic hinge is considered to form in a structural member when the cross section is fully yielded. When a cross section develops a plastic hinge, any addition of moment will cause the beam to rotate with little increase in moment at the plastic hinge location. The fully yielded zone acts as if it were a hinge that can undergo indefinite rotation at a constant restraining moment. Most plastic analysis theories assume that the yield zone is concentrated at zero length plasticity. In fact, the yield zone is developed over a certain length, normally called the hinge length, depending on the loading, boundary conditions and geometry of the section.

Within SAFJAC, plastic hinges are first developed at the sections subjected to the greatest curvature deformation. The possible locations for plastic hinges to develop are at points of concentrated loads, intersections of members involving a change in geometry or points of zero shear for member under uniform distributed load. In analysis of complex forms, it is sometimes convenient to replace at times the non-linear behaviour by an ideal plastic model which yields at a constant stress corresponding to the maximum failure conditions, as shown in Figure 22.

![Figure 22: Ideal bi-linear elastic-plastic approximation](image)

where \( \sigma = \text{stress} \) and \( \varepsilon = \text{strain} \)
SAFJAC incorporates one type of quartic elastic beam-column element (named as Type 34) which allows plastic hinges to form at one or both ends of the element. If requested, the program will automatically subdivide the element into two elements of Type 34 whenever a plastic hinge is detected within the element length.

Studies into the modelling of plastic hinges have concluded that this plastic hinge approach provides only an approximate representation of steel frame behaviour, with its accuracy reducing as the spread of plasticity within the section and along the member becomes important. It is, however, an important tool since it has significant computational advantage over the alternative method of modelling - distributed plasticity [Izzudin and Elnashai, 1993a]. This approach models the buckling behaviour through the inclusion of geometric non-linearities within the quartic formulation, as opposed to modification of the plastic hinge interaction surface. As mentioned above, the plastic hinge formulation is based on quartic elastic formulations and these have eight degrees of freedom. This means that the approach is capable of modelling elastic beam-column elements by the use of only one element per member. Rigid perfectly plastic hinges (with behaviour as shown in Figure 19) are added to the quartic elastic formulation in order to provide an effective method for analysis involving material plasticity.

When plastic hinges occur simultaneously at a node joining two elements the numerical difficulties associated with a diverging solution are likely when the next load step is applied. To overcome this problem, SAFJAC identifies the nodes where the two adjacent plastic hinges may occur and then suppresses one of the hinges by assuming it is rigid for that particular load step [Izzudin and Elnashai, 1993a; Izzudin and Elnashai, 1993b].

4.2.3.2 Foundations

The section of piles inside the legs was modelled by the use of the quartic plastic hinge elements. Lateral movement restraint of the pile to the leg was modelled by spring elements (named as Type 41) which are three-dimensional non-linear joint elements. This joint element is used to connect two nodes by six springs in the local x, y, z, θx, θy, and θz directions. Six different force-displacement relationships are specified to model the axial, shear, torsion and bending properties of the elements (Kx, Ky, Kz, Kθx, Kθy and Kθz). The two nodes to be connected should be coincident. If necessary, a rigid joint element can be defined by specifying large stiffness values [BOMEL, 1992].
Below the mudline, piles were also modelled using quartic plastic hinge elements. The axial response (T-Z curves) and lateral response (P-Y curves) were modelled using spring elements. Five part curves with the ability to define different tensile and compressive behaviour were used to simulate the soil response, as represented in the figure below:

![Generic five-part curve used to describe the force-displacement and moment-rotation of type 41 elements in SAFJAC](image)

**Figure 23: Generic five-part curve used to describe the force-displacement and moment-rotation of type 41 elements in SAFJAC [BOMEL, 1998]**

### 4.2.3.3 Material properties

Bilinear elastic-plastic behaviour is used to represent the material characteristics as hardening occurs. The properties required by the program are Young’s modulus (E), the yield strength (σ_y) and the strain-hardening factor (μ). The material properties used in the model for Leman AP were as follows: Young’s modulus 29,000 ksi, yield strength of 36 ksi for deck columns, jacket legs and braces and yield strength of 50 ksi for jacket leg cans and piles [BOMEL, 1998].

### 4.2.4 Solution derivation

In SAFJAC an incremental iterative solution procedure is adopted. This is based on the so-called frontal technique, which assembles and reduces the global stiffness matrix according to an optimised element order that minimises the bandwidth. The frontal technique is one of a number of methods that have been developed for the solution of large sparse matrices. This technique was refined in the 1980s [Irons and Ahmad, 1980]. A matrix is said to be banded when all the elements are zero except those within a band on either side of the principal diagonal [Astley, 1992]. The semi-bandwidth of such a matrix is the maximum number of terms within the band to the right of (and including) the diagonal. The bandwidth
therefore provides a useful quantitative measure of the compactness of a matrix about its diagonal. The more compact the matrix is, the more efficient the solution process.

The out-of-plane forces are derived for each iteration from the difference between the applied loads and the forces resisted by the structure. These forces are then used in the subsequent iteration and this process is repeated until suitable convergence is attained. However, if convergence is not achieved, or if divergence occurs, then the solution will restart from the last position of equilibrium with a new solution strategy in terms of a new increment or load or displacement. If convergence is not achieved, or if divergence is detected, then SAFJAC will automatically reduce the current load step. Three reduction levels are available which will decrease the step size and which depend on the current level.

In the region of the ultimate load capacity of a structure, a loading increment may not necessarily correspond to an equilibrium configuration. When this occurs convergence cannot be achieved. However, if an appropriate displacement increment is specified, an equilibrium configuration will generally be found. In order to accommodate this, SAFJAC enables the user to control displacements of the structure either by controlling displacements of any node in the global set of axes, or by controlling the rotations of any element.

Both load and displacement control of the solution can be used in order to overcome convergence problems in the vicinity of the ultimate load. The use of automatic scaling of load or displacement increments is another feature of SAFJAC. In load control, the proportional loads are multiplied by a factor which is defined by the user. This gives the loads which are applied (or removed) incrementally according to the number of steps specified by the user. In displacement control, the user defines whether the node translations or rotations are to be monitored and whether the selected displacement is to be increased or reduced by a specific amount. The change is made incrementally according to the number of steps specified by the user.

4.2.5 Code checking of joints

Within the analyses undertaken as part of this research, no checks were performed on the utilisation of tubular joints as required according to API RP2A [API, 1993a]. This would normally be performed as part of a detailed assessment of an installation. The algorithms to perform these standard checks unique to SAFJAC were not made available due to commercial confidentiality. Hence they were not able to be used within this research programme. However, if joints are correctly designed and are not affected by weld defects
or damage, then it is generally considered that members will fail before the joints. Results obtained in this study would be appropriate to those cases where members fail before joints.

4.2.6 Post-processing of results

The output from SAFJAC was manipulated using the post-processing program SAFRES. This extracts information and can be used to produce three different output files. A formatted result file to enable the user to directly examine the results, a graphic interface file and a tabulated file suitable for input into spreadsheets and plot programs. Data derived from SAFRES were then processed further in a variety of ways using a number of different software packages including routines developed as part of this research in Pascal, Microsoft Excel and PATRAN, in order to obtain the following:

- Graphic output of the global load-displacement behaviour.
- Values for the base shear.
- Visual representation of the overall displacement of the whole structure at any load factor of particular interest, i.e. the deformed shape of the structure.
- Determination of the load at which the first member failure occurs in terms of the location of the element where the first plastic hinges occurs.
- Derivation of the dominant failure mode – using the formation and closure of plastic hinges and the load factor and locations at which they occur.
- Derivation of the mobilisation of the soil in the region of the pile in terms of utilisation of the pile capacity – presented in graphical output of capacity compared to actual forces in the pile.

4.2.7 Failure assessment and criteria

In addition to examining the ultimate strength and the load deflection characteristics of the jacket subjected to extreme storm conditions, other indicators were also examined. This was to enable a full picture of the failure loads and modes to be developed. One technique used the examination and location of plastic hinges in both the jacket and the piles. This was used in conjunction with the scrutiny of the mobilisation of the soil in the region of the piles, often expressed in terms of pile utilisation. Another criterion for failure that could be used would involve examination of the load at which the first member fails. This would give a useful indication of the onset of failure of the structure and can also be used to identify the location of the member that is most likely to fail first under storm loading conditions. Structural behaviour beyond first member failure depends on the degree of static indeterminacy, the ability of the structure to redistribute the load and the ductility of individual members. For structures where the first member failure results in system failure,
the probability of collapse is equal to that of first member failure. First member failure was not included in the scope of this project.

In this research, the initial loading is applied to the structural model and then the proportional loading is increased until structural collapse, in order to determine the ultimate load. Ultimate strength or capacity is said to be achieved at the maximum or peak load factor. This can be identified by examination of the load and deflection data and can be seen in the presentation of a load-deflection graph for one, or a number of, carefully selected nodes. Any of the failure assessment techniques mentioned above can be used to derive failure criteria. For example, a value for the maximum global deformation could be set according to serviceability criteria for deck equipment.

In this research, a decision was made to use the peak load as an indication of the ultimate strength. This then formed the basis for the primary failure criteria. The location of the formation of plastic hinges was also examined and was used in the assessment to be made of the failure mode. The number of plastic hinges formed in the jacket (including the riser guards) and in the foundation piles was derived. This was used in association with the degree of mobilisation of the soil in the region of the piles, in terms of pile utilisation. A combination of no plastic hinges forming in the piles, with pile close to being fully utilised was taken as an indication of foundation dominated failure. To confirm this, examination of the deformed shape of the structure at peak load was carried out where necessary. It should be noted that in some cases, when using SAFJAC, an estimated peak load factor had to be used. This was due to the fact that the analysis stopped prematurely due to "excessive sub-increments needed for plastic correction" for a large number of elements, or "excessive iteration to interaction of curve of element".

The overall system effect that was derived from the analysis of detailed structural models was 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.

In this study, the peak load factor was defined as the equivalent of RSR, being the ratio of ultimate to design base shear:

\[
\text{RSR} = \frac{\text{ultimate platform resistance}}{\text{design load}}.
\]
4.3 Yield strength parametric study

4.3.1 Background information

The model selected for the yield strength study was for the wave approach from Platform North. This was selected on the basis that the results showed a clear peak in the load deflection characteristics, that the wave direction was for the end-on condition (i.e. not diagonal loading) and that the analysis time was not excessive. The material properties quoted in the report on Leman AP [BOMEL, 1998] are as follows: Young's modulus 29,000 ksi; yield strength 36 ksi (deck columns, jacket legs, braces) and yield strength 50 ksi (jacket leg cans and piles). Since the model is in SI units throughout, the values entered into the SAFJAC data file were: Young's modulus 0.21 x 10^12 N/m^2, yield strength 0.25 x 10^9 N/m^2 (deck columns, jacket legs, braces) and yield strength 0.35 x 10^9 N/m^2 (jacket leg cans and piles).

4.3.2 Methodology

The mean values of yield strength of 250 MPa (deck columns, jacket legs, braces) and 350 MPa (jacket leg cans and piles) were used. The studies which follow are therefore based around the mean as a starting point for analysis work, but the trends exhibited should be concurrent with those obtained if the values were in fact design values. Variations in yield strength were assigned according to available data from the literature. A COV of 5% was assumed to apply (Frieze et al. [MSL Engineering, 1997] assumed a COV of 4% and [Sigurdsson et al., 1994] assumed a COV of 6%). A lognormal distribution was selected based on previous studies [MSL Engineering, 1997 and WS Atkins, 1997b] and this corresponded to a standard deviation (= mean x COV) of 12.5 MPa and 17.5 MPa respectively.

4.3.3 Summary of results

A summary of the results obtained from the yield strength parametric study is presented here. The subsequent section 4.3.4 examines the results in more detail. Further results are presented in sections 4.3.5 and 4.3.6 for the location of plastic hinges and pile utilisation at peak load respectively. A discussion of the results is then presented in section 4.3.7. Table 27 below shows the results from the pushover analyses performed for the yield strength parameter study. The change in yield strength is shown in percentage terms, followed by the location of the run in terms of the distance from the mean in standard deviations. The actual values of yield strength used in the analysis for the two groups of members (deck columns, jacket legs, braces and then jacket leg cans and piles) is then shown. A range of ±3 standard deviations was selected on the basis that this adequately represents what would
typically be experienced in a fabricated structure. The final column contains the peak load factor derived from the pushover analysis. Peak load factors in this study have been quoted to three decimal places for completeness. It is understood however that this degree of accuracy may not be entirely accurate obtained in such a process.

<table>
<thead>
<tr>
<th>Change in yield strength</th>
<th>Standard deviations from mean</th>
<th>Yield strength values (MPa)</th>
<th>Peak load factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>+15%</td>
<td>+3</td>
<td>287.5 &amp; 402.5</td>
<td>2.058</td>
</tr>
<tr>
<td>+10%</td>
<td>+2</td>
<td>275 &amp; 385</td>
<td>2.004*</td>
</tr>
<tr>
<td>+8</td>
<td>+1.5</td>
<td>268.75 &amp; 373.75</td>
<td>2.013</td>
</tr>
<tr>
<td>+5%</td>
<td>+1</td>
<td>262.5 &amp; 367.5</td>
<td>1.990</td>
</tr>
<tr>
<td>-</td>
<td>-</td>
<td>250 &amp; 350</td>
<td>1.977</td>
</tr>
<tr>
<td>-5%</td>
<td>-1</td>
<td>237.5 &amp; 332.5</td>
<td>1.932</td>
</tr>
<tr>
<td>-8%</td>
<td>-1.5</td>
<td>268.75 &amp; 373.75</td>
<td>1.897</td>
</tr>
<tr>
<td>-10%</td>
<td>-2</td>
<td>225 &amp; 315</td>
<td>1.868</td>
</tr>
<tr>
<td>-15%</td>
<td>-3</td>
<td>212.5 &amp; 297.5</td>
<td>1.805*</td>
</tr>
</tbody>
</table>

Table 27: Summary of runs performed for the yield strength parameter study (*peak not reached)

4.3.4 Examination of results

The load deflection characteristics for all the analyses undertaken were plotted on the same graph, using Node 194 as the reference node studied in the x direction, as shown in Figure 24. This reference node was selected on the basis that its displacement represents the global displacement in the direction of the wave approach. The general shape of the load-deflection curve shows an initial linear portion where an increasing load factor corresponds to an increasing global deflection. After a load factor of around 1.0, the curve starts to exhibit an increasing deflection with increasing load factor. This is due to the commencement of formation of plastic hinges and increasing pile utilisation. Subsequently, a peak is often exhibited before a marked decrease in load with increasing deflection. This occurs mainly for the jacket dominated failure modes, while a smooth load deflection curve is observed for foundation failures.

The run for the +10% case automatically terminated before a distinct peak was exhibited, possibly due to the exceedence of tolerance settings applied in the model set up. The same problem of termination of the analysis before a peak was reached was also exhibited for the run -15%. However, in this case it was found to be due to “excessive sub-increments needed for plastic correction” for a large number of elements and “excessive iteration to interaction of curve of element”. The peak values obtained were therefore assumed to represent close to the peak load factor, but the results were subsequently used with caution.
Figure 24: Load - displacement results for pushover analyses, with different values of yield stress, for the wave approach direction from platform North

Figure 25: Extract of load - displacement results for the wave approach direction from platform North, for load factor greater than 1.5

Figure 24 shows that during the initial stages of loading, up to a load factor of approximately 1.7, all analyses followed the same path. However, after this point, the analyses started to show different characteristics. Those with an increase in the yield strength showed an increased load factor for similar deflections, when compared to the original run with unchanged yield strength, and those with a decrease in yield strength.
showed a decrease in the load factor for similar deflections. Figure 25 shows an enlarged version of Figure 24 for high values of load factor to aid identification.

4.3.5 Plastic hinge locations

An investigation of the failure modes for the different runs was carried out. The number of plastic hinge (PH) locations was used to give an indication of the location and number of elements failing before the ultimate load is reached. Table 28 summarises the runs and the number of plastic hinges formed in different parts of the structure. A way of clearly representing the data pertaining to the location of plastic hinges is to examine the percentage plastic hinges that occurred in the foundations as shown in Figure 26.

![Table 28: Plastic hinges recorded for the yield strength parameter study (*peak not reached)](image)

Figure 26: Plastic hinges formed in the foundation piles in the yield strength study
Figure 26 shows that the number of plastic hinge locations is greatest for the runs with the original yield strength and with a decrease of 5%. The overall trend exhibited from the plastic hinges is interesting. As the yield strength is increased, the number of plastic hinges forming in the foundation piles decreases as the strength in the piles is increased, resulting in a jacket only failure mode. However, as the yield strength was decreased, the failure mode shifted from jacket dominated to mixed. This trend would be an interesting area for the focus of further work.

Along with the number of plastic hinges, the location of the affected elements can also provide a useful insight into the failure mode. Table 29 shows the element numbers of the first five affected members for the cases studied. In addition, study of the location of plastic hinges can be used to create a visual representation of the members that are affected up to peak load. Figure 27 shows side elevation plots for the two rows that form Leman AP, and the location of the elements where plastic hinges were recorded.

<table>
<thead>
<tr>
<th>Change in yield strength</th>
<th>Standard deviations</th>
<th>1st element with PH</th>
<th>2nd element with PH</th>
<th>3rd element with PH</th>
<th>4th element with PH</th>
<th>5th element with PH</th>
</tr>
</thead>
<tbody>
<tr>
<td>+15%</td>
<td>+3</td>
<td>258</td>
<td>257</td>
<td>234</td>
<td>256</td>
<td>688</td>
</tr>
<tr>
<td>+10%</td>
<td>+2</td>
<td>258</td>
<td>257</td>
<td>234</td>
<td>256</td>
<td>688</td>
</tr>
<tr>
<td>+8</td>
<td>+1.5</td>
<td>257</td>
<td>258</td>
<td>234</td>
<td>256</td>
<td>48</td>
</tr>
<tr>
<td>+5%</td>
<td>+1</td>
<td>257</td>
<td>258</td>
<td>234</td>
<td>256</td>
<td>48</td>
</tr>
<tr>
<td>mean</td>
<td>-</td>
<td>234</td>
<td>257</td>
<td>258</td>
<td>256</td>
<td>48</td>
</tr>
<tr>
<td>-5%</td>
<td>-1</td>
<td>257</td>
<td>258</td>
<td>234</td>
<td>256</td>
<td>48</td>
</tr>
<tr>
<td>-8%</td>
<td>-1.5</td>
<td>257</td>
<td>258</td>
<td>234</td>
<td>256</td>
<td>48</td>
</tr>
<tr>
<td>-10%</td>
<td>-2</td>
<td>234</td>
<td>257</td>
<td>258</td>
<td>256</td>
<td>48</td>
</tr>
<tr>
<td>-15%</td>
<td>-3</td>
<td>257</td>
<td>258</td>
<td>234</td>
<td>48</td>
<td>256</td>
</tr>
</tbody>
</table>

Table 29: First 5 elements where plastic hinges (PH) were recorded in yield strength study

From the data presented in Table 29 it can be seen that for all nine cases performed within the yield strength study, only six elements were identified within the first five elements affected by plastic hinges. Only three elements were identified as the first elements affected - elements 258, 257 and 234. It is noteworthy that all these three elements are diagonal bracing elements on the opposite side to the wave approaching from platform North, irrespective of whether the failure was jacket dominated or mixed mode. This shows that the most critical members are represented by elements 234, 256-8 and 48.
4.3.6 Pile utilisation at ultimate load

Investigation into the pile utilisation at peak load for each run was also carried out. A sample pile utilisation plot is shown in Figure 28 for the case with an 8% reduction in yield strength. It can be seen that, for this case, the piles are not all fully utilised. From the plots of pile utilisation against depth of pile, it can be said that with increasing yield strength, there is generally a slight increase in the pile utilisations overall.

For all runs, it was found that pile B row 1 and pile B row 2 exhibited the least utilisation for the wave approach direction studied. The piles in row C, which are on the same side as the wave approach, are in tension, while the piles in rows B and A are in compression. The piles in row B, which is the central row, show a definite decrease in utilisation compared to the piles in rows A and C.

Figure 29 shows the pile layout and wave approach direction.
The following three figures (Figure 30 to Figure 32) show the axial pile utilisation of pile A in row 2, which is under compression loading, for three different cases at yield strength – 5%, unchanged and +5%.

The actual force in the pile is shown compared to the capacity of the pile for both tension and compression cases. It can be seen that as the yield strength is increased, the pile utilisation marginally decreases.
Figure 30: Pile utilisation plot for Pile A Row 2 for case with yield strength reduced by 5%

Figure 31: Pile utilisation plot for Pile A Row 2 for original case (yield strength unchanged)
4.3.7 Discussion of results

The greater the increases in yield strength, the higher the peak load factor exhibited and conversely, the lower the yield strength, the lower the peak load value shown. This can be seen in Table 30 which shows the change in yield strength in percentage and standard deviations, the values used in the model, the peak load factor exhibited, and then the percentage difference between the case with a mean yield strength and the other cases.

<table>
<thead>
<tr>
<th>Increase/decrease in yield strength</th>
<th>Standard deviations from mean</th>
<th>Yield strength values (MPa)</th>
<th>Peak load factor</th>
<th>Difference in peak load from original run</th>
</tr>
</thead>
<tbody>
<tr>
<td>+15%</td>
<td>+3</td>
<td>287.5 &amp; 402.5</td>
<td>2.058</td>
<td>+4.0%</td>
</tr>
<tr>
<td>+10%</td>
<td>+2</td>
<td>275 &amp; 385</td>
<td>2.004*</td>
<td>+1.4%</td>
</tr>
<tr>
<td>+8</td>
<td>+1.5</td>
<td>268.75 &amp; 373.75</td>
<td>2.013</td>
<td>+1.8%</td>
</tr>
<tr>
<td>+5%</td>
<td>+1</td>
<td>262.5 &amp; 367.5</td>
<td>1.990</td>
<td>+0.7%</td>
</tr>
<tr>
<td></td>
<td>-</td>
<td>250 &amp; 350</td>
<td>1.977</td>
<td>-</td>
</tr>
<tr>
<td>-5%</td>
<td>-1</td>
<td>237.5 &amp; 332.5</td>
<td>1.932</td>
<td>-2.3%</td>
</tr>
<tr>
<td>-8%</td>
<td>-1.5</td>
<td>268.75 &amp; 373.75</td>
<td>1.897</td>
<td>-4.1%</td>
</tr>
<tr>
<td>-10%</td>
<td>-2</td>
<td>225 &amp; 315</td>
<td>1.868</td>
<td>-5.7%</td>
</tr>
<tr>
<td>-15%</td>
<td>-3</td>
<td>212.5 &amp; 297.5</td>
<td>1.805*</td>
<td>-9.1%</td>
</tr>
</tbody>
</table>

Table 30: Summary of runs performed for the yield strength parameter study

(*peak not reached)
Figure 33 shows changes in peak load factor with change in yield strength for all analyses, for the yield strength for deck columns, jacket legs and braces and also leg cans and piles.

It was found that linear equations gave a good fit to the two sets of data. The $R^2$ values give an indication of the accuracy of the fit to the actual data (1.0 is an exact fit). It can be seen from the $R^2$ values on Figure 33 that both trendlines represent an acceptable fit to the data. It can be concluded that as the yield strength is increased a proportional increase in the peak load factor was exhibited. As the yield strength was increased from $-3$ to $+3$ standard deviations there was an overall increase in the peak load factor of $\sim 13\%$.

From further examination of the dominant failure mode for each of the runs performed was carried out by a study of the displacement shape of the structure at peak load. The displacement shapes for the runs at $+$ and $-2$ standard deviations are shown in Figure 34 and Figure 35 respectively. The displacements are shown with a graduated colour key. This differs for the two figures and care should be taken when comparing the deflected shapes.

As explained in section 4.3.5, the failure mode for those cases run with increased yield strength were found to be jacket dominated, and those case with decreased yield strength where found to exhibit a mixed mode failure. In the examination of the deflected shapes of the structure this is not as clearly apparent. It is important to note, therefore, that examination of the deformed structure alone may not provide sufficient information on the failure scenario characteristics.
Figure 34: Deformation (m) at peak load for case with yield strength increased by 10%

Figure 35: Deformation (m) at peak load for case with yield strength decreased by 10%
4.3.8 Results from other research

As mentioned in section 4.1.2, the yield strength was studied in detail because it had previously been identified as a key parameter when considering a failure scenario dominated by jacket failure, where possible foundation failure has been ignored. The yield strength was studied in a deterministic manner using structural analysis techniques, in order to assess the sensitivity of the structural response to any changes. The main conclusion from this investigation was that as the yield strength was increased from $-3$ to $+3$ standard deviations there was an overall increase in the peak load factor of $-13\%$. Data from other researchers using a deterministic approach was not available. However, a number of probabilistic studies have been reported with the following approaches.

In a study by [van de Graaf et al., 1994a] it was determined that the COV in member strength could be assumed to be directly related to the COV in yield strength of approximately $10\%$. In subsequent analysis, the lower tail end of the distribution was truncated to reflect rejection of substandard material.

However, in a study by [Sigurdsson et al., 1994] the effect of yield stress on the system capacity of a structure was studied and different conclusions were drawn. The yield stress was assumed to have a normal distribution with a COV of $6\%$. The yield stress within the same structural member was assumed to be fully correlated, but uncorrelated between different structural members. The study concluded that the COV of the system capacity was much less than the COV of the yield stress, in fact approximately $50\%$ lower.

In the 1997 investigation into the reliability of fixed and jack-up structures [MSL Engineering, 1997] the yield strength was modelled using a lognormal distribution with a bias of $1.12$ and a COV of $4\%$. During the study, sensitivity factors were derived using component reliability methods for the jacket and these showed that the yield stress of leg and bracing members contributed between $2\%$ and $4\%$.

4.4 Foundation parametric study

4.4.1 Background information

The model selected for the foundation capacity study was for the wave approach from Platform North. It was the same model as used in the yield strength study, described in the previous section.
4.4.2 Methodology

The foundation springs in the Leman AP are modelled in SAFJAC as element type 41 - these are non-linear spring elements for modelling local flexibility, as described in section 4.2.3.2. A nine parameter (5 part) curve was used to describe the load-deflection behaviour of these pile springs in terms of stiffness, $K$ and deflection, $\Delta$. There are 20 different springs for each pile. The deflections were identified for each of the 20 different springs.

For every spring there are some 54 data points - these relate to three degrees of freedom for each spring. There are 18 points for each degree of freedom - the first 9 relate to tension and the second 9 to compression. In order to study the sensitivity of the foundations, it was decided initially to apply an overall factor to both stiffness and capacity since it was not known whether either or both were a dominant factor. Multiplying the deflections of the springs by a factor enabled both the stiffness and the capacity to be changed.

Once all the load deflection values had been identified, they were then multiplied by a different spring deflection multiplication (SDM) factor for each of the new runs in order to simulate different foundation conditions. By multiplying the deflections and keeping the stiffness the same, the capacity and stiffness of the pile spring elements were changed. A SDM factor of greater than 1.0 gave an increase in the overall capacity and stiffness of the foundations, while a SDM factor of less than 1.0 gave a decrease in the capacity of the foundations and a decrease in the overall foundation stiffness.

Figure 36 and Figure 37 illustrate the changes produced in the load deflection characteristics of the foundation springs. The case shown corresponds to an applied SDM factor of 1.8. Both tension and compression data are shown on the same graph. Figure 38 is for the axial (T-z) data, Figure 39 is for the lateral (P-y) data. The last points shown on each of the graph lines represent the 5th stiffness value and are not a representation of the deflections entered into the model.
Figure 36: Axial T-z data for group 3 elements in the model

The last points on the lines are only to indicate the 5th stiffness value, and are not a representation of the deflections entered into the model.

Figure 37: Lateral P-y data for group 3 elements in the model

The last points on the lines are only to indicate the 5th stiffness value, and are not a representation of the deflections entered into the model.
In order to assess the effect of increasing the foundation stiffness and capacity, runs were performed using SDM factors of 1.3, 1.5, 1.8 and 2. However, the run performed using a SDM factor of 2 exhibited diverging results that exceeded the tolerance and maximum number of iterations set within the model. Even when these were increased, the results did not converge. No results could therefore be produced for the latter case.

In order to assess the effect of decreasing the foundation stiffness and capacity, runs were performed using SDM factors of 0.85, 0.7, 0.6, 0.55 and 0.5. However, the run performed with a SDM factor of 0.5 again exhibited diverging results that exceeded the tolerance and maximum number of iterations set within the model and again, even when these were increased, the results did not converge. An investigation into the utilisation of the piles was also undertaken. Comparison was made between the ultimate capacity and the actual force at peak load, in each of the 20 springs.

### 4.4.3 Summary of results

A summary of the results obtained from the foundation stiffness and capacity parametric study is presented here. The subsequent section 4.4.4 examines the results in more detail. Further results are presented in sections 4.4.5 and 4.4.6 for the location of plastic hinges and pile utilisation at peak load respectively. A discussion of the results is then presented in section 4.4.7. Results from the foundation study are summarised in Table 31, where the peak load factors exhibited for each run are shown. Figure 38 shows the load-displacement plots of the runs.

<table>
<thead>
<tr>
<th>Factor applied to foundation stiffness and capacity</th>
<th>Peak load factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>x 1.8</td>
<td>2.019</td>
</tr>
<tr>
<td>x 1.5</td>
<td>2.006</td>
</tr>
<tr>
<td>x 1.3</td>
<td>2.036</td>
</tr>
<tr>
<td>original</td>
<td>1.977</td>
</tr>
<tr>
<td>x 0.85</td>
<td>1.850</td>
</tr>
<tr>
<td>x 0.7</td>
<td>1.466</td>
</tr>
<tr>
<td>x 0.6</td>
<td>1.242</td>
</tr>
<tr>
<td>x 0.55</td>
<td>1.063</td>
</tr>
</tbody>
</table>

Table 31: Peak load factor results for foundation stiffness and capacity parameter study
1.1.1 Examination of results

The results have shown that where there was an increase in the pile capacity this caused an increase in the peak load factor and where there was a decrease in pile capacity there was also a decrease in the peak load factor. However, as the foundation capacity was increased, only slight increases in the peak load factor were exhibited. The change in peak load factor, therefore, was smaller for those runs with increased foundation capacity than those runs with a decreased foundation capacity. This can clearly be seen from Table 32 where the differences in peak load factor with increases in applied factor is around 0.1, and for decreases in applied factor reaching a decrease around 0.9.

<table>
<thead>
<tr>
<th>Factor applied to foundation stiffness and capacity</th>
<th>Change in foundation capacity</th>
<th>Peak load factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>x 1.8</td>
<td>80%</td>
<td>2.019</td>
</tr>
<tr>
<td>x 1.5</td>
<td>50%</td>
<td>2.006</td>
</tr>
<tr>
<td>x 1.3</td>
<td>30%</td>
<td>2.036</td>
</tr>
<tr>
<td>-</td>
<td>0</td>
<td>1.977</td>
</tr>
<tr>
<td>x 0.85</td>
<td>-15%</td>
<td>1.850</td>
</tr>
<tr>
<td>x 0.7</td>
<td>-30%</td>
<td>1.466</td>
</tr>
<tr>
<td>x 0.6</td>
<td>-40%</td>
<td>1.242</td>
</tr>
<tr>
<td>x 0.55</td>
<td>-45%</td>
<td>1.063</td>
</tr>
</tbody>
</table>

Table 32: Runs performed in the foundation capacity parameter study

Figure 38: Load factor vs. displacement for analyses performed for the foundation stiffness and capacity parameter study
It was found that the load-deformation characteristics could be represented by three main failure modes: jacket dominated, foundation dominated and a mixed failure mode consisting of jacket and foundation failures. Typical load-deflection characteristics are in Figure 39.

![Figure 39: Typical load-displacement plots for the three different failure modes](image)

4.4.5 Plastic hinge locations

The locations of plastic hinges forming during the SAFJAC analysis, up to peak load were studied. Table 33 shows the percentage number of plastic hinges (PHs) forming in the jacket part of the structure (which includes the riser guards) and in the foundations. Figure 40 shows the percentage of plastic hinges formed in the foundations. It can be seen that for the runs performed with a decreased foundation capacity there were no plastic hinges formed within the entire structure up to peak load. This is thought to be due to the 'softening' of the foundations. Indeed, as the foundation capacity was decreased, the analysis produced large displacements with only very small increases in the load applied.

However, it can be seen that as the foundation capacity was increased, an increasing proportion of the plastic hinges formed in the foundations. This was thought to be due to the fact that the mode of failure shifts from being foundation dominated to becoming a mixture of foundation and jacket failure.

The four runs performed with plastic hinges being formed in the jacket and in the foundations all exhibited very similar proportions of plastic hinges. This shows that for this structure, whether the foundation capacity has increased by a factor of 1.3 or 1.8 has very
little effect on the overall load deflection characteristics, both in terms of peak load factor and also in terms of the number and location of plastic hinges.

<table>
<thead>
<tr>
<th>Change in foundation capacity</th>
<th>Factor applied to foundation capacity</th>
<th>Peak load factor</th>
<th>% PHs in jacket (inc. riser guards)</th>
<th>% PHs in foundations</th>
</tr>
</thead>
<tbody>
<tr>
<td>+80%</td>
<td>x 1.8</td>
<td>2.019</td>
<td>66</td>
<td>34</td>
</tr>
<tr>
<td>+50%</td>
<td>x 1.5</td>
<td>2.006</td>
<td>67</td>
<td>33</td>
</tr>
<tr>
<td>+30%</td>
<td>x 1.3</td>
<td>2.036</td>
<td>68</td>
<td>32</td>
</tr>
<tr>
<td>-</td>
<td>-</td>
<td>1.977</td>
<td>69</td>
<td>31</td>
</tr>
<tr>
<td>-15%</td>
<td>x 0.85</td>
<td>1.850</td>
<td>None</td>
<td>None</td>
</tr>
<tr>
<td>-30%</td>
<td>x 0.7</td>
<td>1.466</td>
<td>None</td>
<td>None</td>
</tr>
<tr>
<td>-40%</td>
<td>x 0.6</td>
<td>1.242</td>
<td>None</td>
<td>None</td>
</tr>
<tr>
<td>-45%</td>
<td>x 0.55</td>
<td>1.063</td>
<td>None</td>
<td>None</td>
</tr>
</tbody>
</table>

Table 33: Plastic hinges recorded for the foundation capacity parameter study

![Figure 40](image_url)  

Figure 40: Number of plastic hinges formed in piles, within the foundation capacity study

As mentioned in section 4.4.4, the number of plastic hinges in the generic locations can be used to assess the dominant failure mode of the structure. An additional insight into the characteristics of the structure as it is loaded to peak load, can be obtained by the study of the first members that are affected by plastic hinge formation. Table 34 shows the first five members which have plastic hinges formed, for each of the cases performed in the foundation study. Those cases with decreased foundation capacity and stiffness did not exhibit any plastic hinges. However, when the foundation capacity and stiffness were increased, plastic hinges were formed in six members. Five of these members were diagonal bracing members, whilst only one was a plastic hinge formed in Pile A in Row 1. The precise locations of these members are identified in Figure 42.
It can be seen that all runs had plastic hinges forming in elements 234 and 256-8, and as such these can be concluded to be the most critical members for this wave approach direction. However, it is interesting to note that the fifth member was element 233 for the jacket dominated failure modes, and element 48 for the mixed mode failure.

### 4.4.6 Pile utilisation at ultimate load

Examination of the pile utilisation at peak load shows that, in general, the run performed with decreased foundation capacity showed a higher utilisation than those runs performed...
with increased foundation capacity. This can be seen from the following Figure 42 to Figure 44 showing the pile utilisation of pile A in row 2, under compression loading.

The three figures show the pile utilisation for the cases where a factor of 0.7 was applied to the foundation capacity, the original case and the case where a factor of 1.3 was applied respectively. The utilisation for the x0.7 case (Figure 42) shows the soil is fully mobilised around the pile. The original case (Figure 43) shows that the soil is almost fully utilised all the way down the depth of the pile. The third case where a factor of x1.3 was applied (Figure 44) shows that the utilisation has decreased and the soil is only fully mobilised near to the mudline and not at depth. It can be concluded, therefore, that for Leman AP, as the pile capacity is increased there is an increase in soil mobilisation in terms of pile utilisation.

![Figure 42: Pile utilisation plot for Pile A Row 2 for case where foundation capacity multiplied by a factor of 0.7](image-url)
Figure 43: Pile utilisation plot for Pile A Row 2 for the original foundation case

Figure 44: Pile utilisation plot for Pile A Row 2 for foundation capacity x1.3
4.4.7 Discussion of results

Table 35 shows the change in peak load exhibited, with change in foundation capacity and the difference in peak load factor when compared to the mean (original) case.

<table>
<thead>
<tr>
<th>Factor applied to foundation capacity</th>
<th>Change in foundation capacity</th>
<th>Peak load factor</th>
<th>Difference in peak load from original run</th>
</tr>
</thead>
<tbody>
<tr>
<td>x 1.8</td>
<td>80%</td>
<td>2.019</td>
<td>2%</td>
</tr>
<tr>
<td>x 1.5</td>
<td>50%</td>
<td>2.006</td>
<td>1.5%</td>
</tr>
<tr>
<td>x 1.3</td>
<td>30%</td>
<td>2.036</td>
<td>3%</td>
</tr>
<tr>
<td>-</td>
<td>0</td>
<td>1.977</td>
<td>-</td>
</tr>
<tr>
<td>x 0.85</td>
<td>-15%</td>
<td>1.850</td>
<td>-7%</td>
</tr>
<tr>
<td>x 0.7</td>
<td>-30%</td>
<td>1.466</td>
<td>-30%</td>
</tr>
<tr>
<td>x 0.6</td>
<td>-40%</td>
<td>1.242</td>
<td>-46%</td>
</tr>
<tr>
<td>x 0.55</td>
<td>-45%</td>
<td>1.063</td>
<td>-60%</td>
</tr>
</tbody>
</table>

Table 35: Peak load factor exhibited for foundation capacity parameter study runs

Figure 45 shows change in the foundation capacity with peak load factor. For those runs performed which had a decrease in the overall foundation capacity, the trend could be represented by a second order polynomial. This shows a good fit to the data with an $R^2$ value of 0.99. For those runs performed with an increase in the overall foundation capacity, the trend could also be represented by a second order polynomial fit. This shows an $R^2$ value of 0.57, which is clearly not such a good fit as for the other data. This can be attributed to the fact that an increase of 3% in the peak load factor was exhibited for the run with a SDM factor of 1.3, which was greater than the 2% and 1.5% increase for those runs with a SDM factor of x 1.8 and x 1.5 respectively. The change is only equivalent to 1% but when a trendline is fitted to the data, this will adversely affect the results in terms of the $R^2$ value.

It can also be seen from Figure 45 that the runs with an increase in the pile spring deflections exhibited an increase in the overall stiffness of the load-deflection behaviour of the platform. In contrast, those runs with a decrease in the pile spring deflections exhibited a decrease in the overall stiffness of the load-deflection behaviour. Further investigation of the reasons for the two regions shown in Figure 45, revealed that there were three distinct regions along the curve, in terms of dominant failure mode. The initial region where there is a linear relationship between foundation capacity and peak load factor was found to be foundation dominated. The final region where there is a ‘plateau’ on the trendline, where increases in foundation capacity were not reflected by increases in peak load factor, was found to be jacket dominated. The region which exhibits a curve was found to produce a mixed mode failure, consisting of both foundation and jacket failure as shown in Figure 46.
Further examination into each of the three failure scenarios was performed by study of the displacement shapes of the structure at peak load. The displacements for each of the three failure scenarios are shown in Figure 47, Figure 48 and Figure 49.
Figure 47: Deformation (m) at peak load for a sample foundation dominated failure

Figure 48: Deformation (m) at peak load for a sample mixed mode failure
The colour key for the deformation in the three figures differs, and care should be taken when making comparisons. However, it can be seen that in the case of the foundation failure, there are significantly greater deformations in the piles compared to the jacket-dominated failure, which exhibits very little deformation. In the mixed mode case, the deformation in the piles is less than that in the foundation-dominated case but more than in the jacket-dominated failure case.

In all three cases there has been significant deflection of the riser guards, although this should not be of great significance to the overall performance of the whole structure. This is because the structure was designed and installed to withstand the 50-year extreme environmental condition, and then at a later date, the riser guards were added with the sole purpose of protecting the risers from ship impact.
4.5 Concluding remarks

When examination of the failure mode is undertaken, it is important that not just one method is applied. The information provided by the deflected shape alone may not necessarily provide sufficient insight into the dominant failure mode. It is best that a combination of methods are used, such as examination of the deflected structural shape at peak load, the location and number of plastic hinges, in conjunction with interrogation of the extent of pile utilisation or soil mobilisation.

The main conclusion to be drawn from the yield strength parametric study is that a linear relationship exists between the peak load factor exhibited and the yield strength of the structural material. This means that as the yield strength is increased, a proportional increase in the ultimate capacity will be exhibited. The changes are fairly small, but as the yield strength is increased from $-3$ to $+3$ standard deviations, it can be concluded that there is an overall increase in the ultimate load of $\sim 13\%$ for the specific structure studied. Further studies would be needed on other structures to establish whether this trend is exhibited by other platforms.

The principal finding to be drawn from the foundation capacity parametric study is that when foundation stiffness and capacity are reduced, then a significant decrease in the peak load factor is noted. This shows a relationship that can best be predicted with a polynomial curve. However, at some point, as the foundation capacity is increased, then no apparent increase in peak load factor is observed. The changes in sensitivity with foundation capacity were related to the dominant failure mode. Three regions were identified in terms of failure mode: jacket only, foundation only and mixed mode failure.
CHAPTER 5.
THE EFFECT OF KEY PARTS OF THE PROCESS: FOUNDATION ASSESSMENT

5.1 Introduction

As described in the previous chapter, three main areas for further work were identified through the development of the framework. Briefly, these areas were:

- examination of the effect of certain individual parameters used within the overall reliability analysis process
- scrutiny of the effect of key parts of the process and
- analysis of the different methods of reliability assessment.

This chapter examines the second issue.

In Chapter 2, foundation capacity and reliability were identified as an important issue in structural system reliability assessment. Due to the fact that foundations have not been previously examined in detail within the context of system capacity and reliability of fixed offshore platforms, the research here has focused on this area for a detailed investigation.

This chapter, therefore, examines the large uncertainties that can be associated with foundation reliability and capacity. It is necessary to focus effort on establishing how key factors of the assessment process can affect the overall structural reliability assessment. Aspects such as soil type, soil profile, assessment method, and inclusion of potentially beneficial or detrimental effects can all affect the overall foundation assessment.

Uncertainty in prediction of foundation behaviour and soil-structure interaction arises from the following: spatial variation in soil properties, limited site exploration, limited calculation models, uncertainties in soil parameters and also uncertainties in load. The load-displacement behaviour of piles in sand under axial loading will depend on a variety of factors including: the profiles with depth of limiting shaft resistance, the variations of the shear strength of the sand with shear strain and mean effective stress, the effective axial
stiffness of the pile, the conditions at the pile base (plugged, unplugged or closed) and the sense of the loading (tension or compression) [Bond et al., 1997].

As described in Chapter 2, research has shown that there are three main factors that can effect the capacity and stiffness of the foundations – these are the effects of cyclic loading, rate of loading and ageing [Poulos, 1988]. One important feature of offshore pile foundations is the cyclic nature of the loading, both axial and lateral. To date, limited experimental investigations have been carried out and an extensive testing programme is currently ongoing at Imperial College in association with WS Atkins [Jardine, 1999]. Tests to date generally indicate that ‘two-way’ cyclic loading, which involves pile reversal, may have a significant effect in reducing pile capacity and stiffness, whereas ‘one-way’ cyclic loading has a smaller effect.

Another factor to be considered when assessing piled foundations is the rate of loading. This can potentially increase both the load capacity and pile stiffness due to the rapid rate of load application caused by relatively high frequency of wave loading. Tests have shown that for piles in sand, there is little or no effect of loading rate on either skin friction or pile head stiffness. However, for piles in clay the rate of load application has a significant effect on pile load capacity, which can be of the order of 10 to 20% [Poulos, 1988].

In addition, time can affect the foundation capacity. Preliminary work from Imperial College has shown an increase in the foundation axial capacity, by a factor of approximately 2, for piles examined more than 1 year after installation, with no apparent change in the foundation base capacity.

In this research, the effects of cyclic loading and ageing on the capacity of piles in sand have been studied in detail and are described in the following sections. Rate of loading has not been included in this research, as the effect that it has on piles in sand is negligible.

5.1.1 Scope of work

Of the 200 or so offshore platforms installed in the UK sector of the North Sea, over 90% are pile-supported steel jackets and hence this study was performed using Leman AP which is of this type [Bond et al., 1997]. Smaller jacket structures are founded on one pile per leg, as is the case with the Leman AP structure, whereas larger jackets can be supported with typically 4-8 piles in each group. Piles themselves are usually open-ended steel pipes up to 2m in diameter and 80m or more in length.
This chapter deals with a series of studies undertaken to scrutinise the effect of key parts of the reliability assessment process on the results. The foundations were identified as the main area for uncertainty within the overall reliability assessment and one that has not been previously examined in depth. Here one aim was to explore the effect of using different methods to assess the capacity of the foundations.

The most common procedure is to apply the API RP2A assessment procedure. However, more recently, significant work on offshore piling has been undertaken at Imperial College (IC), London [Jardine and Chow, 1996b; Jardine et al., 1998] where new design approaches have been developed for analysing driven piles in clays and sands. The new deterministic methods were found to be relatively simple and easy to apply in practice; and were 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” [Jardine and Chow, 1996b].

The API foundation assessment methodology has been shown to be conservative for sands and unconservative for some clay soil types and, furthermore there is a large COV associated with its use. In order to move towards a more accurate assessment within a reliability analysis, it is necessary to use the best available method and hence comparisons are made between the API and IC methods.

5.1.2 Assessment of soil types in the North Sea

The stratigraphy of the North Sea is complex. It consists, in most areas, of some combination of sands and clay. Sands are present in almost all parts of the North Sea as a thin surface layer, interbedded with clays, or as the predominant soil type. The North Sea can be split into three regions according to soil type. In the northern North Sea, stiff to very stiff over-consolidated clays are found, although in many areas these are interbedded with dense fine sand. In the central North Sea, interbedded clays and sands predominate, whereas in the south there is a large tract of mainly fine to coarse sand. The abundance of sand in the UK sector of the North Sea was quantified by Fugro-McClelland in 1983, and the proportions of platforms that are founded on the different soil profiles are shown in Figure 50 [see Bond et al., 1997]:

From Figure 50 it can be deduced that between 54% and 69% of all piled foundations rely on sand providing some part (if not the majority) of their capacity. It was for this reason that the investigation described in the following sections was based on a soil type of North
Sea sand. Leman AP is located in the southern North Sea where North Sea sands are dominant [Bond et al., 1997].

There are two main parameters that affect the foundation capacity of piles that are driven in sand: ageing effect and cyclic loading effect. Current work at IC is investigating the precise nature of long-term ageing effects for piles in sands. Preliminary work has shown an increase in the foundation capacity by a factor of approximately two for piles examined more than one year after installation [Jardine, 1999]. If piles are exposed to cyclic loading, then this effect can degrade the foundation capacity. WSA Atkins and IC [Jardine, 1999] are currently investigating the effects of cyclic loading in detail.

When comparing the API pile capacity assessment method and the newer IC method for piles in sand, the API method is found to be significantly over-conservative for piles in sand [Jardine and Chow, 1996a]. Thus moving from using the API method to the IC method results in an increase in the value of the resistance of the foundations, as well as a reduction in the associated COV. Both these factors contribute to an increase in reliability between API based foundation assessments and IC based assessments.

Piles driven in clay soils do not necessarily exhibit the same characteristics as similar piles driven in sands. Indeed, the effects of ageing and cyclic loading appear to be substantially less than for piles in sand. For piles in clay, however, the rate of load application has a
significant effect on pile load capacity. The variation can be in the order of 10 to 20% [Poulos, 1988].

Using the API method of assessment and comparing it to the newer IC method for piles in clay shows that the API method is less conservative than the IC method - this implies a potentially negative effect on the overall system reliability. However, a reduction in COV is incurred when using the IC assessment method, which implies a positive effect on the overall system reliability. The type of clay found at a particular location would have an influence on which has the greater effect, and hence which dominates the overall reliability assessment. Thus, the overall effect on system reliability may be positive or negative. In a recent MSL study of one platform in clay soils a negative effect was found [MSL Engineering, 1997]. Further work is needed to investigate all these foundation effects over a wider range of parameters.

5.1.3 Foundation capacity assessment methods

A number of assessment methods have been developed to formulate an estimate of foundation capacity [NGI, 1994a; Hamilton and Murff, 1995; Cherubini, 1997; Jardine and Chow, 1996b; and Jardine et al., 1998.]. In the past, the most widely adopted method has been the API recommended practice. More recently, however, the new assessment method developed by IC is becoming increasingly utilised. The bases of the two methods are discussed in the following sections.

The API practice is based on the assumption that the radial effective stress at the point of shaft failure is a function of the vertical effective stress. It also incorporates the assumption that the interface angle of friction is a function of the grain size and relative density. Recent work at IC has found that evidence suggests that in fact the interface angle of friction is not proportional to the relative density. The IC formulations developed were based on the assumption that the radial effective stress acting on the shaft at failure is dependent upon the value acting after installation, the pore pressure and the radial stress equalisation, combined with any changes developed during pile loading. Recent research at IC 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 API gives recommended practice for assessment of piled foundations [API, 1993a]. It provides the following formula for determining the ultimate shaft resistance of a single pile in sand:

$$\tau_s = K \sigma'_v \tan \delta + \tau_{\text{max}}$$

Where: $$\tau_s$$ = ultimate shear resistance of a single pile  
$$K$$ = dimensionless earth pressure coefficient  
$$\sigma'_v$$ = original vertical effective stress in the ground  
$$\delta$$ = angle of interface friction between the sand and the pile wall  
$$\tau_{\text{max}}$$ = limiting value of shaft friction

A number of different values for $$K$$, $$\delta$$, and $$\tau_{\text{max}}$$ have been introduced since 1969 when the recommendations were first produced. Changes have been triggered by the results of pile tests. Changes to the earth pressure coefficient made in the 15th edition were not universally adopted, since it appeared that the values quoted were unconservative for piles installed in North Sea sands. Indeed, the 19th edition included a warning that the existing recommendations may not be entirely reliable. A so-called North Sea variant therefore emerged in UK practice based on the values described in the 15th edition for both $$\delta$$ and $$\tau_{\text{max}}$$. However, instead of using the API values for $$K$$ where it was suggested that it was appropriate to assume $$IC = 0.8$$ for both tension and compression loading, the North Sea variant adopted values of $$IC = 0.7$$ for compression and $$K = 0.5$$ for tension [Hobbs, 1993b].

As described in the review study detailed in Chapter 2, IC developed new methods for the assessment of piles in sand and clay [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:

$$\tau_f = \sigma'_{rf} \tan \delta_f$$

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

Thus the radial effective stress acting on the shaft at failure depends on the value acting after installation and full pore pressure and radial stress equalisation, combined with any changes developed during pile loading. The $$\tan \delta_f$$ 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 local pile shaft capacity over the external pile area [Jardine and Chow, 1996b].
Large-scale tests were performed to investigate the effects of time on the capacity. Results showed that the shaft capacity increased by \(-85\%\) between 6 months and 5 years. However, no comparable gains were found for base resistance. The effect of time were therefore represented as follows:

\[
\frac{Q_s(t)}{Q_s(t = 1 \text{ day})} = 1 + A \left[\log\left(\frac{t}{t = 1 \text{ day}}\right)\right]
\]

Where:
- \(Q_s\) = shaft capacity
- \(t\) = time of assessment (up to a maximum of five years)
- \(A\) = coefficient (value is \(0.5 \pm 0.25\))

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 is the value of \(\sigma'\), developed at failure, and differs slightly from \(\sigma'_{re}\) the equilibrium value by acting prior to loading. Pile installation and subsequent equalisation lead to \(\sigma'_{re}\) values that usually exceed "free-field" horizontal effective stress \(\sigma'_{ho}\) where \(\sigma'_{re}\) can vary considerably during the potentially lengthy equalisation period. It should be noted that the API recommendations do not take into account any of the above features, but instead, use a total stress approach for calculating shaft friction [Bond et al., 1997].

5.2 The effect of soil profile and assessment method for piles in sand

5.2.1 Introduction

The Leman field site consists of marine sands, which are prone to movement on the seabed and whose density can vary rapidly with location and depth. A re-assessment of the site was carried out in 1983 and a report submitted to Amoco. Pile driving records were also given for piles driven at other locations in the Leman field [Jardine, 1999]. Based on these data, it was assumed that the Leman AP profile consisted entirely of sands over the depth of interest, with at least two distinct layers of denser and looser sand. Within these were large potential variations of CPT resistance. Silty or clayey layers were found at some locations, but these were not noted in the Leman AP logs and were thus not included in the analysis.
Cone penetration test (CPT) data from other Leman platform locations (operated by AMOCO and Shell) held by IC were used to formulate a set of possible CPT profiles appropriate for the AP location.

The problem posed was to generate a set of possible CPT profiles which would be representative of conditions that might reasonably be expected at the site. The first profile produced was synthesised such that it represented an overall safety factor on the structure of 1.5 when the API assessment method was adopted. This was selected on the basis that the API method would have been used when the installation was designed and installed, with a safety factor of at least 1.5. Three other profiles were then developed around this data.

Figure 51: Cone CPT (MPa) with depth below top of clay layer (m) for the four soil profiles derived
Figure 51 shows the four soil profiles derived for the study by IC. A typical median or ‘design’ profile (Profile I) was derived based on multiple CPT soundings from other Leman structures. For more detailed information, two structures belonging to Shell are described in detail in [Jardine et al., 1998]. A second profile (named Profile II) was specified in which the initial layer was considered looser than in Profile I. In assessing the profile, credible minima and maxima were assessed based on the full set of CPT data.

Two further profiles (Profiles III and IV) were specified to help assess the possible variations in the soil profile. Profile III was considered to be denser, and hence more favourable, in terms of capacity, while Profile IV was looser and likely to give lower axial capacities. Profiles III and IV are approximately one standard deviation above and below the baseline design Profile I. These four profiles formed the basis of the soils profiles adopted for quantitative pile capacity assessments.

The sands at the location were considered to be ‘very dense’. The relative density can be derived by a variety of methods, and there is no consistent industry approach. Indeed, differences exist between different codes and different countries. Lloyds Register currently advocates the use of the ‘Baldi method’ and it was therefore decided to adopt this method [Lunne et al, 1997] within this research.

5.2.2 Case studies using the IC method

Case 1 was based on a vertical pile, with a step change in CPT profile at 15.5m depth, using design median (Profile I). Case 2 was based on a vertical pile with a step change in CPT profile at 10m depth, using data from Leman BD (Profile II). Case 3 was based on a vertical pile, with a step change in CPT profile at 15.5m depth, using an ‘unfavourable’ profile (Profile IV). Case 4 was based on a vertical pile, with a step change in CPT profile at 7m, using a ‘favourable’ (Profile III). Cases using the IC method assumed global scour of 2m.

The existence of possible clay layers was ignored. An assessment made with the IC method’s plugging criterion showed that the conditions at the pile tip were close to the margin where they could either plug during static compressive loading, or fail with a coring action. The piles were therefore assumed unplugged. Incidentally, if plugging had been assumed, a slight increase in the total capacity would have been predicted.
5.2.3 Case studies using the API method

Cases I to IV were run without changes to the piles or assumed soil profiles, but applying the API North Sea variant method instead of the IC method. All cases using the API method assumed global scour of 2m. Again, the existence of possible clay layers was ignored. Using the API criteria, piles were assumed to exhibit fully plugged ends during static compression loading.

5.2.4 Analysis cases run

A total of eight cases were run in order to assess the effect of changing the soil profile. Four cases were run for both the IC and API assessment methods. Both IC and API capacities were derived assuming that cyclic loading had no effect on capacity and that there was no need to allow for any additional gains in capacity as a result of ageing.

There were eight cases to be analysed. These are as follows as shown in Table 36:

<table>
<thead>
<tr>
<th>Cases to be analysed</th>
<th>Profile description</th>
</tr>
</thead>
<tbody>
<tr>
<td>API Case 1</td>
<td>Profile I ‘design’</td>
</tr>
<tr>
<td>API Case 2</td>
<td>Profile II</td>
</tr>
<tr>
<td>API Case 3</td>
<td>Profile IV ‘unfavourable’</td>
</tr>
<tr>
<td>API Case 4</td>
<td>Profile III ‘favourable’</td>
</tr>
<tr>
<td>IC Case 1</td>
<td>Profile I ‘design’</td>
</tr>
<tr>
<td>IC Case 2</td>
<td>Profile II</td>
</tr>
<tr>
<td>IC Case 3</td>
<td>Profile IV ‘unfavourable’</td>
</tr>
<tr>
<td>IC Case 4</td>
<td>Profile III ‘favourable’</td>
</tr>
</tbody>
</table>

Table 36: Cases to be studied to investigate the effect of changing the soil profile

A comparison was made of the data input into the SAFJAC model, for each of the above cases. For the API cases, it was found that despite the profiles being altered, only slight variations in axial capacity were derived. In particular, when profiles III and IV were assessed by the API method, there was no change in the axial capacity and hence only one analysis (API Case 3) was performed. This is due to the fact that the API method is relatively insensitive to changes in sand density once sand has been classified as ‘very dense’.

Figure 52 shows the total axial pile capacity for compression loading for each of the eight cases studied. It can be seen that the compression capacity for API Case 3 is the same as API Case 4. It also shows that the API method generally predicts slightly greater compression axial capacity than the IC method.
Figure 53 shows the total axial pile capacity for tension loading for each of the eight cases studied. It can be seen that for the tension case the API method predicts lower axial capacity than the IC method.

![Figure 52: Total axial pile capacity (corrected for scour) for the eight cases studied for compression loading (data supplied by IC)](image1)

![Figure 53: Total axial pile capacity (corrected for scour) for the eight cases studied for tension loading (data supplied by IC)](image2)
Irrespective of the profile adopted, the API capacity predictions were largely linear with pile depth, with only profiles 2 and 3 showing a step change where the density of the soil was significantly changed, and where this affected the axial capacity. The shear stress is plotted against the depth of the pile in Figure 54 showing the compression loading case and in Figure 55 showing the tension loading case.

![Figure 54: Maximum shear stress against depth of pile (m) for API assessed cases 1, 2 and 3 for compression loading (data supplied by IC)](image)

![Figure 55: Maximum shear stress against depth of pile (m) for API (North Sea variant) assessed cases 1, 2 and 3 for tension loading (data supplied by IC)](image)
For the IC cases, the change in profile directly influenced a change in the axial capacity. Where the profiles had a step change according to a change in the density of the soil, the capacity also exhibited a step change. The characteristics of the axial capacity with depth down the pile are shown in Figure 56 and Figure 57.

![Figure 56: Maximum shear stress against depth of pile (m) for IC assessed cases for compression loading (data supplied by IC)](image)

![Figure 57: Maximum shear stress against depth of pile (m) for IC assessed cases for tension loading (data supplied by IC)](image)
In general, the trend is non-linear, with increasing capacity with increasing depth, for both the tension and compression load cases. The profile that gives the largest axial capacity overall is IC 4, which uses the ‘favourable’ profile 3.

Conversely, the profile that gives the least capacity overall is IC 3, which uses the ‘unfavourable’ profile 4. The ‘design’ profile (IC 1) can be seen, in general, to predict a higher capacity than the ‘unfavourable’ case (IC 3) and a lower capacity than the ‘favourable’ case (IC 4).

5.2.5 Results

The results from the study into the effect of changing the soil profile are presented here. These results are discussed later in section 5.2.6.

5.2.5.1 Load displacement characteristics

Table 37 shows the results obtained for each of the analysis runs, for peak load factor, and the percentage difference when compared to the ‘design’ profile results.

<table>
<thead>
<tr>
<th>Cases analysed</th>
<th>Profile used</th>
<th>Peak Load Factor</th>
<th>% difference compared to ‘design’ profile results</th>
</tr>
</thead>
<tbody>
<tr>
<td>API Case 1</td>
<td>Profile I ‘design’</td>
<td>2.020</td>
<td>-</td>
</tr>
<tr>
<td>API Case 2</td>
<td>Profile II</td>
<td>2.016</td>
<td>-0.2%</td>
</tr>
<tr>
<td>API Case 3</td>
<td>Profile IV ‘unfavourable’</td>
<td>1.984*</td>
<td>-1.8%</td>
</tr>
<tr>
<td>IC Case 1</td>
<td>Profile I ‘design’</td>
<td>2.006</td>
<td>-</td>
</tr>
<tr>
<td>IC Case 2</td>
<td>Profile II</td>
<td>1.979</td>
<td>-1.4%</td>
</tr>
<tr>
<td>IC Case 3</td>
<td>Profile IV ‘unfavourable’</td>
<td>2.045*</td>
<td>1.9%</td>
</tr>
<tr>
<td>IC Case 4</td>
<td>Profile III ‘favourable’</td>
<td>2.001</td>
<td>-0.2%</td>
</tr>
</tbody>
</table>

* Load deflection results did not exhibit a clear peak

Table 37: Peak load factor results for different profiles for IC and API methods

Load-displacement characteristics of the analyses performed are shown in Figure 58 for the API cases and Figure 59 for the IC cases. It should be noted that API Case 3 and IC Case 3 did not exhibit a clear peak in the load deflection characteristics.
Figure 58: Load-deflection characteristics of analyses performed using the API method (the ‘original’ run here was that initially used from data supplied in the model).

Figure 59: Load-deflection characteristics of analyses performed using the IC method (The ‘original’ run here was that initially used from data supplied in the model.)
5.2.5.2 Plastic hinge formation results

A study of the plastic hinges formed during each of the analysis runs was undertaken. This assessment involved extracting data pertaining to the location (element and node number), load factor and status of the plastic hinge. The percentage of plastic hinges are shown in Table 38 for each analysis. The percentage of plastic hinges in the jacket, riser guards and foundations are identified, and can be used to assess the dominant failure mode.

<table>
<thead>
<tr>
<th>Run name</th>
<th>Jacket</th>
<th>Riser guard</th>
<th>Foundations</th>
</tr>
</thead>
<tbody>
<tr>
<td>IC Case 1</td>
<td>76%</td>
<td>24%</td>
<td>0%</td>
</tr>
<tr>
<td>IC Case 2</td>
<td>85%</td>
<td>15%</td>
<td>0%</td>
</tr>
<tr>
<td>IC Case 3</td>
<td>84%</td>
<td>16%</td>
<td>0%</td>
</tr>
<tr>
<td>IC Case 4</td>
<td>82%</td>
<td>18%</td>
<td>0%</td>
</tr>
<tr>
<td>API Case 1</td>
<td>78%</td>
<td>22%</td>
<td>0%</td>
</tr>
<tr>
<td>API Case 2</td>
<td>88%</td>
<td>13%</td>
<td>0%</td>
</tr>
<tr>
<td>API Case 3</td>
<td>92%</td>
<td>8%</td>
<td>0%</td>
</tr>
<tr>
<td>API Case 4</td>
<td>92%</td>
<td>8%</td>
<td>0%</td>
</tr>
<tr>
<td>Original case</td>
<td>70%</td>
<td>20%</td>
<td>10%</td>
</tr>
</tbody>
</table>

Table 38: Percentage of plastic hinges formed up to peak load in each location

Figure 60 shows the number of plastic hinges formed, up to peak load, grouped by profile adopted. The results from the original model have been included for comparison only.

![Figure 60: Plastic hinges formed up to peak load, grouped by assessment method](image-url)
5.2.6 Discussion of results

5.2.6.1 IC assessed cases

For those cases assessed using the IC method, cases 1, 2 and 4 exhibited a clear peak in the load-deflection characteristics. However, Case 3 did not exhibit a peak, and the analysis was terminated after excessive iterations were encountered. Further analysis was undertaken in order to assess whether the last output was near to the peak and the last result was taken to be equivalent to the peak load factor. Cases 1, 2 and 4 behaved in a similar manner, up until close to the peak load. Case 1 showed a peak, followed by a gradual ‘unloading’, whilst Cases 2 and 4 showed distinct and marked ‘unloading’.

The differences in peak load exhibited by the IC assessed cases, when compared to the ‘dile case (Case 1), were -1.4%, 1.9% and -0.2% for Cases 2, 3 and 4 respectively. These changes are larger than those exhibited by the API assessed cases. The load factor is seen to decrease (when compared to Profile 1) when Profile 2 is adopted, to further decrease for Profile IV (unfavourable) and then to decrease still further for Profile III (favourable). The latter result was not expected and it was thought that the ‘favourable’ profile would provide more foundation capacity, and hence a higher peak load, than the ‘unfavourable’ profile. This was found to be due to the fact that the analysis for Case 3 did not exhibit a clear peak.

From the study of the plastic hinge locations in conjunction with the load deflection characteristics it was concluded that all the IC assessed cases were jacket-dominated failures. This shows that despite the changes to the soil profile, the foundations are still of sufficient capacity to ensure that failure does not occur in the foundations.

5.2.6.2 API assessed cases

For the three cases run in conjunction with the API assessment method, the load-deflection characteristics are very similar. The initial part of the loading curve shows only slight change between Case 2 and Cases 1 and 3. As loading progresses, the load-deflection characteristics become very close, and the three runs are virtually indistinguishable when compared in Figure 58. The result for the peak load was taken as the point of inflection for API Case 1. For API Case 2, the analysis had just exhibited a peak before excessive iterations were required and the analysis was terminated. For API Case 3, no peak was exhibited and further analysis needs to be undertaken in order to assess whether the last output was near to the peak. At this stage, however, the last result was taken to be equivalent to the peak load factor.
The percentage differences in peak load exhibited by the API cases, when compared to the ‘design’ profile case (Case 1), were -0.2% and -1.8% for Cases 2 and 3 respectively. These changes are small and there are two possible explanations for this. Firstly, the failure mode of these three cases has shown that no plastic hinges formed in the foundations. This is an indication that the failures are jacket dominated. This would mean that despite slight changes to the axial capacity of the foundations, very little effect on the overall peak load factor is produced. The other possible explanation stems from the fact that the API assessment method is relatively insensitive to relative density and hence, once a certain level is exceeded, it has no extra effect on the foundation capacity. This is why there was no difference in the axial capacity derived for API Cases 3 and 4. This, therefore, could be the reason that the API assessed cases appear relatively insensitive to the different profiles.

From the study of the plastic hinge locations in conjunction with the load deflection characteristics it was concluded that, like the IC cases, all API assessed cases were also jacket-dominated failures. This shows that despite the changes to the foundation soil profile, or changes to the assessment method, the foundations were still of sufficient capacity to ensure that failure did not occur in the foundations, but in the jacket of the structure.

5.3 The effect of cyclic loading and ageing for piles in sand

5.3.1 Introduction

As mentioned at the beginning of the chapter there are two main issues that affect piles in sands: namely, the effect of cyclic loading and the effect of ageing. The effect of cyclic loading is potentially detrimental to the overall axial pile capacity, whilst the effect of ageing is potentially beneficial to the overall axial pile capacity.

A series of analyses were undertaken to investigate different combinations of these two effects. For the cyclic loading, the effect was studied by simply investigating analyses run ‘with’ and ‘without’ cyclic loading. For the effect of ageing, two cases were again studied by simply investigating either the short-term case that would exhibit no ageing effect, or the long-term case that would exhibit the beneficial ageing effect.

5.3.2 Effect of cyclic loading on piles in sand

Based on the preliminary results from ongoing testing, Imperial College recommended that a degradation of capacity in the order of 15-20% would be a reasonable assumption for a well-designed platform with a safety factor of 1.7 to 1.8. However, if the safety factor was
nearer 1.5, the potential degradation of capacity could be greater [Jardine, 1999]. A more accurate prediction of the effect of cyclic loading could have been made if data for the cyclic loading of Leman 49/27 AP had been available. The storm loads would have to be idealised into a number of cyclic blocks, each characterised by a minimum pile head load, a maximum pile head load and the number of cycles applied. However, such data were not made accessible to this study and therefore, in order to represent the effect of cyclic loading a degradation of 20% has been adopted for this research, in terms of both axial and lateral capacity, as well as end bearing capacity.

5.3.3 Effect of ageing on piles in sand

When taking into account the effect of ageing, the aspects that need to be considered include the timing of potential storm events and their likelihood of occurrence. Current field tests by Imperial College have indicated enhanced capacity with time and it was considered reasonable to assume that after 50 days, a 50% increase in capacity would be likely [Jardine, 1999]. This 50% increase after 50 days would be appropriate if the API North Sea variant assessment method were to be adopted. However, with the IC method, this increase would occur in a shorter time, approximately 10 days after driving, rather than after 50 days. Capacity gains would continue to take place for some months, stabilising perhaps one year after installation. The static capacity developed by the time the first serious storm loading event took place would probably be greater than that calculated by the IC method. Tests have shown that this increase due to ageing affects shaft capacity but has little or no effect on base capacity. Therefore, in order to represent the effect of ageing in this research, an increase of 50% on only the shaft foundation capacity has been assumed, in terms of both axial and lateral capacity.

5.3.4 Analysis cases run

The assessment of the effects of cyclic loading and ageing has been in two parts. The first set of analyses undertaken was based on the ‘design’ profile assessed with the IC method. The failure mode of the structure produced when the ‘design’ profile was applied was a jacket-dominated failure. An overall ‘blanket’ factor was applied to all the axial, lateral and end bearing capacities. This was justifiable on the basis that in jacket-dominated failure, end bearing will have very little or no influence on the failure, irrespective of the fact that ageing only affects axial and lateral shaft capacity. Details of the analyses undertaken on the structure when a jacket-dominated failure mode was exhibited are shown in Table 39.
In order to assess the effect of cyclic loading and ageing on a structure that had a mixed mode of failure (i.e. both foundation and jacket), a second set of analyses were undertaken. These were based on the ‘design’ profile, assessed by the IC method, but with an overall decrease in the axial, lateral and end bearing capacities of a factor of 0.3.

The details of the analyses undertaken on the structure when a mixed mode failure was exhibited are shown in Table 40. In each analysis set, in order to assess both the effect of cyclic loading and the effect of ageing, four analysis cases were run. These were performed with and without ageing, and with and without cyclic loading. The details of these analyses are as follows, where an indication of the effect on foundation capacity is represented by + and – signs to show the expected overall effect of the changes under consideration:

<table>
<thead>
<tr>
<th>Cases analysed</th>
<th>Effect on foundation capacity</th>
<th>Quantitative change to foundation capacity</th>
<th>Overall factor applied to foundation capacity</th>
</tr>
</thead>
<tbody>
<tr>
<td>No ageing, no cyclic</td>
<td>- +</td>
<td>0%, 0%</td>
<td>no change</td>
</tr>
<tr>
<td>No ageing, with cyclic</td>
<td>- -</td>
<td>0%, -20%</td>
<td>x0.8</td>
</tr>
<tr>
<td>Ageing, no cyclic</td>
<td>+ +</td>
<td>+50%, 0%</td>
<td>x1.5</td>
</tr>
<tr>
<td>Ageing, with cyclic</td>
<td>+ -</td>
<td>+50%, -20%</td>
<td>x1.3</td>
</tr>
</tbody>
</table>

Table 39: Description of the first set of cases to be studied to investigate the effect of ageing and cyclic loading for jacket dominated failure of the structure

<table>
<thead>
<tr>
<th>Cases analysed</th>
<th>Effect on foundation capacity</th>
<th>Change to foundation capacity</th>
<th>Factor on foundation capacity</th>
<th>Factor on axial &amp; lateral capacity</th>
<th>Factor on end bearing capacity</th>
</tr>
</thead>
<tbody>
<tr>
<td>No ageing, no cyclic</td>
<td>- +</td>
<td>0%, 0%</td>
<td>no change</td>
<td>0.7</td>
<td>0.7</td>
</tr>
<tr>
<td>No ageing, with cyclic</td>
<td>- -</td>
<td>0%, -20%</td>
<td>x0.8</td>
<td>0.56</td>
<td>0.56</td>
</tr>
<tr>
<td>Ageing, no cyclic</td>
<td>+ +</td>
<td>+50%, 0%</td>
<td>x1.5</td>
<td>1.05</td>
<td>0.7</td>
</tr>
<tr>
<td>Ageing, with cyclic</td>
<td>+ -</td>
<td>+50%, -20%</td>
<td>x1.3</td>
<td>0.91</td>
<td>0.56</td>
</tr>
</tbody>
</table>

Table 40: Description of second set of cases to be studied to investigate the effect of ageing and cyclic loading for mixed mode failure of the structure

5.3.5 Results

The results from the study into the effect of cyclic loading and ageing are presented here. These results are discussed later in section 5.3.6.

5.3.5.1 Effect of ageing and cyclic loading on jacket-dominated failure

The peak load factor results from the first four cases analysed are shown in detail in Table 6. The case letter allocated to each run shown is also used in Figure 61 for clarity.
<table>
<thead>
<tr>
<th>Cases analysed</th>
<th>Case</th>
<th>Change to foundation capacity</th>
<th>Factor on foundation capacity</th>
<th>Change in foundation capacity</th>
<th>Peak L.F.</th>
<th>% difference in peak L.F.</th>
</tr>
</thead>
<tbody>
<tr>
<td>No ageing, with cyclic (- -)</td>
<td>[B]</td>
<td>0%, -20%</td>
<td>x0.8</td>
<td>-20%</td>
<td>1.983</td>
<td>-0.6%</td>
</tr>
<tr>
<td>No ageing, no cyclic (- +)</td>
<td>[C]</td>
<td>0%, 0%</td>
<td>no change</td>
<td>0%</td>
<td>1.995</td>
<td>0.0%</td>
</tr>
<tr>
<td>Ageing, with cyclic (+ -)</td>
<td>[D]</td>
<td>+50%, -20%</td>
<td>x1.3</td>
<td>30%</td>
<td>2.028</td>
<td>1.6%</td>
</tr>
<tr>
<td>Ageing, no cyclic (+ +)</td>
<td>[E]</td>
<td>+50%, 0%</td>
<td>x1.5</td>
<td>50%</td>
<td>2.046</td>
<td>2.5%</td>
</tr>
</tbody>
</table>

Table 41: Ageing and cyclic loading effect results for jacket-dominated failure of structure

Figure 61 shows the load deflection characteristics of the four analyses undertaken based on jacket-dominated failure:

![Load deflection characteristics](image)

Figure 61: Load deflection characteristics exhibited for the ageing and cyclic analyses based on jacket-dominated failure.

Figure 62 then shows the percentage difference results against percentage change in foundation capacity, for the set of analyses based on a jacket-dominated failure. It can be seen that only slight changes to the peak load factor are exhibited, despite the different foundation capacities used.
Figure 62: Change in peak load factor against change in foundation capacity for the four ageing and cyclic analyses based on jacket-dominated failure.

<table>
<thead>
<tr>
<th>Comparison</th>
<th>Effect of ageing</th>
<th>Effect of cyclic</th>
<th>Extreme cases</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(no cyclic)</td>
<td>(with cyclic)</td>
<td>(no ageing)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>(with ageing)</td>
</tr>
<tr>
<td>Difference</td>
<td></td>
<td></td>
<td>B &amp; E</td>
</tr>
</tbody>
</table>

Table 42: Percentage difference results for the effect of ageing and cyclic loading for the first set of analyses based on jacket-dominated failure.

It can be seen that for this structure even when the worst case scenario (short term, with cyclic) is compared to the best case scenario (long term, no cyclic), the change in peak load factor exhibited is ~3%.

5.3.5.2 Effect of ageing and cyclic loading on mixed mode failure of the structure

The peak load factor results from the four cases analysed based on a mixed mode failure are shown in detail in Table 43. Figure 63 shows the load deflection characteristics of analyses based on mixed mode failure that includes the case letters shown in the table.

<table>
<thead>
<tr>
<th>Cases analysed</th>
<th>Case</th>
<th>Change to foundation capacity</th>
<th>Factor on axial capacity</th>
<th>Factor on end bearing capacity</th>
<th>Peak L.F.</th>
<th>% diff. in peak L.F.</th>
</tr>
</thead>
<tbody>
<tr>
<td>No ageing, with cyclic (- -)</td>
<td>[B']</td>
<td>0%, -20%</td>
<td>0.56</td>
<td>0.56</td>
<td>1.435</td>
<td>-30.2%</td>
</tr>
<tr>
<td>No ageing, no cyclic (- +)</td>
<td>[C']</td>
<td>0%, 0%</td>
<td>0.7</td>
<td>0.7</td>
<td>1.946</td>
<td>-</td>
</tr>
<tr>
<td>Ageing, with cyclic (+ -)</td>
<td>[D']</td>
<td>+50%, -20%</td>
<td>1.05</td>
<td>0.7</td>
<td>2.022</td>
<td>+3.8%</td>
</tr>
<tr>
<td>Ageing, no cyclic (+ +)</td>
<td>[E']</td>
<td>+50%, 0%</td>
<td>0.91</td>
<td>0.56</td>
<td>2.007</td>
<td>+3.1%</td>
</tr>
</tbody>
</table>

Table 43: Peak load factor results for ageing and cyclic loading effect runs, for mixed failure mode of the structure.
Figure 63: Load deflection characteristics exhibited by the four ageing and cyclic analyses, based on a mixed mode base case (IC x0.7)

Figure 64 shows the percentage difference results against percentage change in foundation capacity, for the set of analyses based on a mixed mode failure. It can be seen that more significant changes to the peak load factor are exhibited, than those that were exhibited for the jacket-dominated failure.

Figure 64: Change in peak load factor against change in foundation capacity for the four ageing and cyclic analyses based on mixed mode failure.
Structural System Reliability Framework For Fixed Offshore Platforms

<table>
<thead>
<tr>
<th>Comparison</th>
<th>Effect of ageing (no cyclic)</th>
<th>Effect of ageing (with cyclic)</th>
<th>Effect of cyclic (no ageing)</th>
<th>Effect of cyclic (with ageing)</th>
<th>Extreme cases (-- &amp; ++)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Difference</td>
<td>+3.1%</td>
<td>+34.0%</td>
<td>-30.2%</td>
<td>+0.7%</td>
<td>33.2%</td>
</tr>
</tbody>
</table>

Table 44: Percentage difference results for the effect of ageing and cyclic loading for the first set of analyses based on jacket-dominated failure

It can be seen that for this structure, when the worst case scenario (short term, with cyclic) is compared to the best case scenario (long term, no cyclic), the change in peak load factor exhibited is ~33%.

5.3.6 Discussion of results

5.3.6.1 The effect of ageing and cyclic loading on jacket-dominated failure

The results for the peak load have been taken as the point of inflection for runs B and E that did not exhibit clear peaks. The peak load factors for runs C and D were taken as the maximum load factor at the first peak. The dominant failure mode for each run has been derived from examination of the location of plastic hinge formation and from the degree of utilisation of the soil around each of the six piles. It was found that for each run, jacket failure was the dominant failure mode, with no plastic hinges being formed in the piles.

From the data derived, the following initial conclusions may be drawn. It would seem that:

- The average effect of ageing is +2.4% on the peak load factor exhibited.
- The average effect of cyclic loading is −0.7% on the peak load factor exhibited.
- The difference between the “worst case” and “best case” is 3.1%.

The influences of the effect of ageing and the effect of cyclic loading in this case are small. Results have been derived to one decimal place, and it is valid to ask whether the procedures adopted can in fact be this accurate.

Since the failure of the structure is jacket dominated, increase and decrease in foundation capacity has produced only small changes in the overall peak load factor. However, the trends show that the effect of ageing is to increase the overall structure capacity, whilst the effect of cyclic loading is to decrease the overall capacity. This trend becomes clearer in the following set of results.

5.3.6.2 The effect of ageing and cyclic loading on mixed mode failure

The peak load factors for runs B', C', D' and E' were taken as the maximum load factor at the first peak. All four runs exhibited a clear peak load. The dominant failure mode for
each run was derived from examination of the location of plastic hinge formation and from the degree of utilisation of the soil around each of the six piles.

It was found that for the ‘base’ case [C’] there was a mixed mode of failure i.e. both jacket and foundation failed at peak load, with domination of the jacket failure mode. For the worst case run (- -) which represented the case with no ageing, but with cyclic loading, there were no plastic hinges formed up to peak load, (or indeed afterwards). Examination of the pile utilisation indicated a foundation-dominated failure. The run for the best case (+ +), which represented the case for ageing with no cyclic effect, exhibited a failure mode which was dominated by the formation of plastic hinges in the jacket (and riser guards). For the other intermediate run (+ -), which represented the case for ageing with cyclic loading, jacket failure was again the dominant failure mode, with no plastic hinges being formed in the foundation piles.

From the data described above, the following initial conclusions (which are specific to the platform considered here) may be drawn:

- The effect of cyclic loading, when ageing is not present, is a decrease of 30.2% on the peak load factor exhibited. However, the effect of cyclic loading, when ageing is present, is a marginal increase of 0.74% on the peak load factor exhibited.
- The effect of ageing, when cyclic loading is not present, is an increase of 3.1% on the peak load factor exhibited. However, the effect of ageing, when cyclic loading is present, is an increase of 34.0% on the peak load factor exhibited.
- The difference between the “worst case” and “best case” is 33.2%.

The influence of the effect of ageing would seem to be distinct and measurable when taking the mixed mode failure as the starting ‘base’ case. However, the effect of cyclic loading is not so tangible for the cases with ageing (+ - and + +) as these cases showed jacket-dominated failure, and hence are not as sensitive to changes in the foundation capacity.

5.4 Further investigations into parameter sensitivity

In order to move towards improved foundation reliability predictions, the best available method of assessment must be used. The trends noted in the earlier parametric studies due to the effects of changing the foundation characteristics were revisited. The effects of changes to the foundations were therefore assessed using the IC method, with application of the ‘design’ soil Profile I.
The initial parametric study into the effects of changing the foundation capacity and stiffness were performed using the original model with application of a factor to both the capacity and the stiffness. In this part of the research, the effect of changing only the stiffness was studied in order to observe whether it had a significant effect on the peak load factor exhibited. Two additional runs were performed, using the IC ‘design’ profile I as the base case, but with changes to the axial and lateral stiffness of the foundation by doubling or halving the stiffness values. Table 45 shows the details and results for these runs.

<table>
<thead>
<tr>
<th>Change in stiffness</th>
<th>Peak load factor</th>
<th>Change in peak load factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case 1 (++) double</td>
<td>1.979</td>
<td>+0.1%</td>
</tr>
<tr>
<td>Original</td>
<td>1.977</td>
<td>0</td>
</tr>
<tr>
<td>Case 2 (- -) half</td>
<td>1.937</td>
<td>-2%</td>
</tr>
</tbody>
</table>

Table 45: Summary of runs performed for the foundation stiffness study based on the IC assessment method and ‘design’ profile I

The load deflection characteristics of these three cases have been studied. It was found that the peak load factor exhibited negligible change with stiffness increase or decrease. The case run with double the stiffness exhibited stiffer behaviour overall, when compared to the original run or the run with half the stiffness. Figure 65 shows the load displacement characteristics for the three runs considered in the foundation stiffness study.

![Figure 1: Load - displacement results for pushover analyses using the IC assessment method with different values of foundation stiffness](image)
It can be concluded therefore that although changing the stiffness of the foundations had an effect on the overall stiffness of the structure, it did not have any significant effect on the ultimate load of the structure. Therefore, in the next analyses undertaken, only the capacity of the foundations was changed. Eight runs had already been performed in this way, for the ageing and cyclic loading assessments, and as a result, only one additional run was performed. This was used to investigate whether the trends observed in the original foundation parametric study in Chapter 4 were exhibited in this part of the work using the IC assessment method. The runs that incorporated the effect of ageing had to be factored in order to obtain an overall change to the foundation capacity despite the fact that the effect of ageing was not applied to the end bearing.

Table 46 shows a summary of analyses performed to investigate the effect on peak load factor of changing the foundation capacity. It can be seen that an additional run [A] was performed in order to provide additional information on the trend of the peak load factor with changing foundation capacity. The peak load factor versus percentage change to the foundation capacity is shown in Figure 66.

It can be seen that the trend observed for the initial parametric study (where both the capacity and the stiffness were changed by the same proportion) was similar to the trend identified when only the foundation capacity was changed using the IC assessment method. The trend is very similar in the fact that, as the foundation capacity is increased beyond a certain point, no increase in the peak load factor is produced. However, as the foundation capacity is reduced, a greater decrease in the peak load factor is found.

<table>
<thead>
<tr>
<th>Case letter</th>
<th>Effect on capacity</th>
<th>Overall change to foundation capacity</th>
<th>Peak load factor</th>
<th>Difference in peak load factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>[A]</td>
<td>- -</td>
<td>-55%</td>
<td>0.819</td>
<td>-84%</td>
</tr>
<tr>
<td>[B']</td>
<td>-</td>
<td>-44%</td>
<td>1.435</td>
<td>-33%</td>
</tr>
<tr>
<td>[C']</td>
<td>- +</td>
<td>-30%</td>
<td>1.946</td>
<td>-2.5%</td>
</tr>
<tr>
<td>[B]</td>
<td>-</td>
<td>-20%</td>
<td>1.983</td>
<td>-0.6%</td>
</tr>
<tr>
<td>[D']</td>
<td>+ -</td>
<td>-19.5%</td>
<td>2.022</td>
<td>1.3%</td>
</tr>
<tr>
<td>[E']</td>
<td>+ +</td>
<td>-5.5%</td>
<td>2.007</td>
<td>0.6%</td>
</tr>
<tr>
<td>[C]</td>
<td>- +</td>
<td>0%</td>
<td>1.995</td>
<td>0%</td>
</tr>
<tr>
<td>[D]</td>
<td>+ -</td>
<td>30%</td>
<td>2.028</td>
<td>1.6%</td>
</tr>
<tr>
<td>[E]</td>
<td>+ +</td>
<td>50%</td>
<td>2.046</td>
<td>2.5%</td>
</tr>
</tbody>
</table>

Table 46: Summary of analyses performed to investigate the effect on peak load factor of changing the foundation capacity

It should be noted that examination of the output data, as with the initial parametric study, showed that those analyses that were performed with increased foundation capacities
produced jacket-dominated failure. Similarly, those runs performed with decreased foundation capacities exhibited mixed mode and then foundation-dominated failure as the foundation capacity was further decreased.

It can be seen that the initial slope derived for the two sets of data are different (see Figure 66). This may be due to differences in the soil profile adopted. Further investigation into this would be required in order to confirm this, however, it is outside the scope of work for the current research.

![Figure 66: Peak load factor exhibited for runs with different foundation capacity](image)

**5.5 Concluding remarks**

Investigations into the assessment of foundation capacity, one of the factors affecting reliability have been undertaken in some detail. Relevant background information has been presented along with details of the individual studies performed. An examination of the issues surrounding foundation capacity assessment has been undertaken, which has focussed on three key areas where the effect on the ultimate capacity of the structure has been studied for different soil types, soil profiles and capacity assessment methods. Additional aspects included in the IC assessment method have also been studied including analysis of the effects of cyclic loading and ageing, on both a jacket-dominated failure scenario and a mixed mode failure scenario.
The studies undertaken have been performed on a six-legged structure in the southern North Sea, where the soil type assumed was ‘very dense’ North Sea sands. The analyses have been undertaken for one wave approach direction – that from platform North. Any conclusions drawn therefore relate only to these conditions and further work would be required in order to affirm the trends and results noted for other structures, in additional locations, with different soil conditions. On this basis, the following conclusions can be made:

When the four different soil profiles were adopted and the API assessment method was used to predict foundation axial capacity, it was found that despite changes to the soil profile very little change was exhibited in the predicted capacity. However, when the IC method was used, significant differences in the predicted capacity were noted. This is because the API method is insensitive to any changes in soil profile once the soil has been classified as ‘very dense’ whereas the IC method is sensitive to soil profile due to the way in which it predicts the capacity.

When comparison was made between the API and IC assessment predictions of axial foundation capacity, based on the same soil profile, it was found that under compression loading the results were similar, but with API predicting marginally larger capacities. However, for the soil profiles adopted which were synthesised as ‘unfavourable’ and ‘favourable’ it was found that the IC method predicted a decreased capacity for the ‘unfavourable’ condition and a substantial increase in capacity for the ‘favourable’ condition.

The effects of cyclic loading and ageing were assessed for piles in sand. For a jacket-dominated failure mode it was found that negligible changes to the ultimate capacity of the structure were exhibited. This was due to the fact that the foundations were sufficient to ensure that failure still occurred in the jacket, despite the changes caused by cyclic loading or ageing. However, when the structure with a mixed mode failure was studied (i.e. both foundation and jacket failure) it was found that the detrimental effect of cyclic loading, with no allowance for ageing, changed the failure mode to foundation dominated. This corresponded with a significant decrease in the ultimate capacity of the structure being predicted. When the positive effect of ageing was applied with no cyclic loading a change in the peak load factor was observed, with an increase in peak load exhibited compared to the original case. This case exhibited a jacket-dominated failure mode.
When the foundation stiffness was studied, it was found that negligible changes to the peak load were exhibited and it was therefore concluded that foundation capacity dominated the foundation characteristics and its effects on the structural system behaviour. When foundation capacity was studied, it was seen that the same trend was exhibited as that derived in the preliminary parametric study. As the foundation capacity decreased, a significant decrease in the peak load factor was attained, but when the capacity was increased beyond a certain point, no further increase in peak load factor occurred. This is due to a shift from foundation-dominated failure, through mixed mode failure into a region dominated by jacket failure, where any increase in foundation capacity does not correspond to an increase in the peak load factor. The findings will be used in Chapter 6 as the basis for the work on response surface methodology and will also enable improvements to be incorporated into the revised framework presented in Chapter 7.
CHAPTER 6.

SYSTEM RELIABILITY
ANALYSIS METHODS

6.1 Introduction

Through the development of the framework, as described in Chapter 3, three main areas for further work were identified. Briefly, these areas were: examination of the effect of certain individual parameters used within the overall reliability analysis process, scrutiny of the effect of key parts of the process and analysis of the different methods of reliability assessment. This chapter addresses the third issue of assessing the different reliability analysis methods. The reliability analysis methods applied in this research are the 'minimal' analysis and the response surface techniques. A third method has been compared, the system analysis approach developed and applied by WSAtkins [Shetty, 1994]. The level of detail presented in this chapter attempts to provide sufficient information on each of the three methods in order that comparisons may be made. For each method, the approach is to identify the key aspects of each method, provide a summary of intermediate results, describe the method of derivation of reliability index and probability of failure and make a comparison with other relevant investigations. A simplified system analysis approach is introduced, and a new approach for preliminary reliability estimation is suggested. However, further work would be needed to develop this further and validate this approach for a range of structures and wave approach directions.

6.1.1 Basis for study

Detailed information on the evaluation of structural reliability of offshore platforms was presented in Chapter 4. It was explained that quasi-static analyses are conducted on an offshore structure with dynamic effects incorporated by inclusion in the inertial load set approach. To conduct platform ultimate strength analyses to establish failure, the loading must be increased beyond that corresponding to the applied load. Thus, the environmental loading pattern is increased using a load factor until the ultimate strength is reached. Beyond this peak load factor point, the strength will soften, either gradually or rapidly depending on the mode of failure and the behaviour of the members and foundations.
The term ‘pushover analysis’ is widely used within the offshore industry to describe three-dimensional, non-linear, large displacement, static finite element analysis. The horizontal forces on a platform structure, representing ocean wave forces, are increased until structural collapse is reached. The failure and post-failure behaviour of platform components, such as brace members and tubular joint connections, are modelled explicitly. The underlying assumptions in a static pushover analysis are firstly that the failure of an offshore platform structure occurs when a single large wave strikes the platform and secondly that the wave period is sufficiently longer than the natural period of the structure so that dynamic effects are negligible. In most cases for fixed offshore platforms, these assumptions are reasonable since wave forces dominate the structural loading and the natural frequency of such a structure is generally in the region of about four times less than the period of the applied design / ultimate ocean wave [Dawson, 1993].

6.1.2 Scope of work

Through the literature review and as described in detail in Chapter 3, there are three main techniques used to perform structural reliability assessments. These are the minimal analysis approach, the response surface technique and the system analysis approach. These methods have been applied to one structure namely, Leman AP. Analyses were undertaken as part of this research using the minimal analysis and response surface techniques. Confidential information was provided by the research sponsors related a system analysis study that had been undertaken on the same structure.

As previously mentioned, one method of deriving the most dominant failure paths is to perform a pushover analysis. The most critical elements are identified in this analysis, but no account is made of the effect of possible variations in component strength that could result in different sequences of failure and different combinations of elements. Monte Carlo simulation is a method for obtaining information about system performance from component data, which has 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. The effects of these variations can be explored more fully using simulation techniques. However, the use of such methods is limited in the case of offshore platforms because of the large size and scale of the problem under consideration. Such simulations were utilised and reported by Sigurdsson et al. (1994) and Shell Research (1993). However, no studies using the numerical simulation approach have been performed on Leman AP and no comparison of results could be made. Furthermore, the approach is not considered to be a practical alternative for the present application.
6.2 Procedure adopted for reliability analysis

6.2.1 Introduction

Reliability methods deal with the uncertain nature of loads and resistance and are based on analysis models for the structure under consideration. The analysis models are usually imperfect and the information about load and resistance is usually limited. In the past, the reliability was generally taken as a nominal measure of safety of the structure, given a certain analysis model, and was dependent on the amount and quality of information. Measuring the safety of a structure by its reliability therefore provided a useful decision variable. More recently advances in modelling of the load, structure and resistance have enabled the industry to move towards more consistent and 'true' reliability and it is in this context that this research is significant.

Structural system reliability analysis is concerned with the reliability assessment of structures exhibiting multiple failure modes. The probability of occurrence of each failure mode contributes to the overall failure probability for the structure and each failure mode may involve a complex scenario with failure of several structural members in a sequence. Reliability analysis results must be interpreted critically as the results and the corresponding conclusions significantly depend on the analysis model and associated bias, along with the distribution types adopted [DNV, 1992]. The measure of reliability is most usually taken as the reliability index, which is defined as a function of the probability of failure. Additional results that may also be derived from a reliability analysis are sensitivity factors (parameter sensitivities and importance factors) and an estimation of the most likely failure point called the design point.

6.2.2 Probabilistic analysis

In this research, the probabilistic analysis was performed using the software PROBAN (PROBabilistic ANalysis) developed by DNV [DNV, 1989]. This is a general program for probabilistic, reliability and sensitivity analysis. Features of interest are that it incorporates first and second order reliability methods (FORM/SORM) and it contains an extensive statistical distribution library. If necessary, PROBAN also contains the necessary features to perform Bayesian updating and parameter studies. PROBAN can be executed via an interactive graphical user interface and the results can be presented graphically. The results can be produced in terms of probability of failure, probability distribution, importance of each uncertain variable and sensitivity with respect to model parameters, e.g. mean value/standard variation. In this research, PROBAN was mounted on a SUN workstation.
and operated from UNIX. Variables within PROBAN can be modelled as numeric constants, functions, distributions, time dependent stochastic variables and probability of events. A parameter of a variable can be assigned a co-ordinate of another variable so that a network structure for dependencies between variables can be defined. Additional statistical dependence between the variables can be modelled through correlation.

6.3 Application of the ‘minimal’ analysis approach in this research

6.3.1 Methodology

When the ‘minimal’ analysis technique is applied, pushover analyses are used to determine the dominant failure modes for each direction and develop failure surfaces. Such pushover analyses were performed for this research using the non-linear finite element software package SAFJAC and an existing model of the Leman AP platform. Four different wave approach directions were studied: waves from platform West, North West, North and South East. [For a more detailed explanation of the model, see Chapter 3.]

Environmental design data for the 50-year extreme storm condition, in terms of wave height, period and current were supplied with the model. Maximum base shears and maximum overturning moment for all eight compass directions for the 50-year condition were also provided. As described earlier, loading on the structure was applied in two stages. Firstly, the initial loading effectively for the still water condition (including wind on the topside) was applied and secondly, the proportional loading for the environmental loading pertaining to the 50-year extreme storm condition was used. Reliability analysis was then performed using the software package PROBAN, using the first order reliability method. The reliability index, $\beta$ and the probability of failure were derived for each case. The higher the reliability index, the more reliable the system; and the higher the probability, the higher the likelihood of failure. In the reliability analyses, the 50-year design extreme storm condition base shear was used to derive the mean load assuming a lognormal distribution and the ultimate base shear was taken as the mean resistance, assuming a lognormal distribution.

6.3.2 Summary of intermediate results

Results obtained from the pushover analyses for the four wave approach directions studied are shown in Table 47. The table includes the 50-year design base shear values supplied, along with the ultimate base shear results obtained. Figure 67 shows the 50-year design base shear plotted alongside the ultimate base shear results on a compass diagram.
Models pertaining to all wave approach directions were not made available, and only four were studied: platform W, NW, N and SE.

It can be seen that the design and ultimate base shear results differ with different wave approach directions. This is due to a combination of two factors: the first being that different environmental conditions are present in the different wave approach directions, and the second due to differences in structural configuration of the platform.

<table>
<thead>
<tr>
<th>Wave approach direction</th>
<th>50 yr. design base shear (MN)</th>
<th>Ultimate base shear (MN)</th>
<th>1</th>
</tr>
</thead>
<tbody>
<tr>
<td>Platform W</td>
<td>8.11</td>
<td>14.05</td>
<td></td>
</tr>
<tr>
<td>Platform NW</td>
<td>7.97</td>
<td>13.24</td>
<td></td>
</tr>
<tr>
<td>Platform N</td>
<td>6.04</td>
<td>12.26</td>
<td></td>
</tr>
<tr>
<td>Platform NE</td>
<td>5.71</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>Platform E</td>
<td>6.89</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>Platform SE</td>
<td>6.76</td>
<td>12.85</td>
<td></td>
</tr>
<tr>
<td>Platform S</td>
<td>3.44</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>Platform SW</td>
<td>5.51</td>
<td>-</td>
<td></td>
</tr>
</tbody>
</table>

1 - analysis performed as part of this research project

Table 47: Design and ultimate base shear results derived using SAFJAC results for Leman AP

Figure 67: Compass plots of 50-year design base shear and ultimate base shear for Leman AP
6.3.3 Derivation of reliability index and probability of failure

Calculations were performed in order to ascertain the value for mean load and mean resistance. The loading distribution under investigation was taken and normalised in order that it corresponded to the standard normal distribution to allow standard normal tables to be used. The return period was taken as the 50-year extreme storm condition. The probability, $P$, of encountering the 50-year extreme storm condition in any one year was taken as follows:

$$P = \frac{1}{\text{return period}} = \frac{1}{50} = 0.02$$

The definition of the coefficient of variation, $COV$, was:

$$COV = \frac{\text{standard deviation}}{\text{mean}} = \frac{\sigma}{\mu}$$

For the lognormal distribution case, a random variable $X$ has a logarithmic normal probability distribution if $\ln X$ (the natural logarithm of $X$) is normal. A value for the COV of the loading was assumed to be 0.2, based on the literature studied [WSAtkins, 1997b].

The equations for a lognormal distribution are:

$$\log L = N(\mu, \sigma^2)$$

Mean, $\mu = e^{\mu + \frac{1}{2} \sigma^2}$

Variance, $\text{Var.} = e^{2\mu + \sigma^2}(e^{\sigma^2} - 1)$

$$COV = \frac{e^{\frac{1}{2} \sigma^2} \sqrt{e^{\sigma^2} - 1}}{e^{\frac{1}{2} \sigma^2}}$$

Hence the load relating to the 50-year return condition can be derived from the following:

$$L_{50} = \Phi \left( \frac{(\log L_{50} - \mu)}{\sigma} \right) = 0.98$$

When the standard normal distribution tables are examined, the value for $x$ corresponding to $\Phi(x)$ of 0.98 is found to be 2.06.

In order to derive values for mean and standard deviation the two equations for COV and standard normal distribution function were solved simultaneously by an iteration technique. This was carried out using the Solver analysis tool within Microsoft Excel.
In general, a lognormal distribution is a better assumption than a normal distribution for both the resistance and loading variables [Barltrop et al., 1993]. This distribution is defined where the logarithms of the resistance and loading are normally distributed.

For lognormally distributed loading and resistance, the reliability index can be estimated as follows:

$$\beta = \frac{\mu_{\ln R} - \mu_{\ln L}}{\sigma_{\ln R-L}} = \frac{\mu_{\ln R} - \mu_{\ln L}}{\sqrt{\sigma_{\ln R}^2 + \sigma_{\ln L}^2}} = \frac{\ln \left( \frac{\mu_R}{\mu_L} \right)}{\sqrt{\left( \frac{\sigma_R}{\mu_R} \right)^2 + \left( \frac{\sigma_L}{\mu_L} \right)^2}}$$

where: $\mu_R$ = mean resistance, $\mu_L$ = mean loading
$\sigma_R$ = standard deviation of resistance, $\sigma_L$ = standard deviation of loading

It should be noted that this expression provides only an estimate of the reliability index, but it is often used in practice because it is conveniently written in terms of the mean and the standard deviation of the actual variable. However, another more precise formula can be written in terms of the mean and standard deviation of the logarithm of the variable. A textbook expression for the 'exact' formula to derive the reliability index was also used for when the load, L and the resistance, R, were both lognormally distributed. This had previously been used in a review of structural reliability based criteria for fixed offshore platforms [Efthymiou et al., 1996], where detailed reliability assessments were undertaken.

The 'exact' expression is given by the following expression:

$$\beta = \ln \left( \frac{\mu_R}{\mu_L} \right) \sqrt{\frac{1 + V_R^2}{1 + V_L^2}}$$

where: $\mu_R$ = mean resistance, $\mu_L$ = mean loading
$V_R$ = COV of resistance, $V_L$ = COV of loading

Figure 68 shows the predictions derived using PROBAN, with the estimated and 'exact' predictions for Leman AP, for one wave direction - that from platform North. This used a sample loading COV of 0.30.
It can be seen that for this case, the PROBAN results and the ‘exact’ prediction follow similar characteristics and represent an approximately linear prediction. However, the estimate would seem to over-predict the reliability index, particularly at the lower resistance COVs.

The next procedure was to identify the two critical wave approach directions. From the results obtained, these directions were for waves approaching from platform North West and West respectively. Assuming independence of system failure events for different wave approach directions, the overall failure probability was estimated thus:

$$\text{Overall } P(f) = P(f)_1 + P(f)_2 - P(f)_1 \cdot P(f)_2$$

Where $P(f)_1$ and $P(f)_2$ are the probability of failure for different wave approach directions.

Results of the overall probability of failure for Leman AP derived from reliability analyses are shown in Table 48:

<table>
<thead>
<tr>
<th>Wave</th>
<th>$P(f)$ ($R_{COV}$ 0.15, $L_{COV}$ 0.2)</th>
<th>$P(f)$ ($R_{COV}$ 0.15, $L_{COV}$ 0.3)</th>
<th>$P(f)$ ($R_{COV}$ 0.15, $L_{COV}$ 0.35)</th>
</tr>
</thead>
<tbody>
<tr>
<td>P-NW</td>
<td>1.34E-04</td>
<td>8.16E-05</td>
<td>1.52E-05</td>
</tr>
<tr>
<td>P-W</td>
<td>7.83E-05</td>
<td>6.03E-05</td>
<td>1.30E-05</td>
</tr>
<tr>
<td>Overall</td>
<td>2.12E-04</td>
<td>9.68E-05</td>
<td>7.33E-05</td>
</tr>
</tbody>
</table>

Table 48: Overall probability of failure for Leman AP from reliability analyses derived in this research
From the results shown in Table 48, it can be seen that as the COV of the loading is increased, while the COV of the resistance remains unchanged, that the probability of failure decreases. The overall probability of failure derived from the probability of failure for the two wave approach directions therefore decreases with increasing loading COV.

6.3.4 Comparison with results from other investigators

Results from two other minimal analysis studies were obtained relating to the same structure, Leman AP. The first was performed by WSAtkins [WSAtkins, 1997b] and the second by Amoco, quoted in [WSAtkins, 1997b]. The WSAtkins reliability analyses identified the two critical wave approach directions as those from Platform North West and Platform North and using their values, gave an overall probability of failure of 2.04E-04.

The Amoco results quoted by WSAtkins, again identified the two critical wave approach directions as those from Platform North West and Platform North, but gave an overall probability of failure as 9.30E-05 [WSAtkins, 1997b]. The results are shown in Table 49.

<table>
<thead>
<tr>
<th>Analysis by</th>
<th>Software</th>
<th>Load parameter</th>
<th>Resistance parameter</th>
<th>Overall P(f)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Author¹</td>
<td>SAFJAC</td>
<td>Lognormal COV 0.2</td>
<td>Lognormal COV 0.15</td>
<td>2.12E-04</td>
</tr>
<tr>
<td>WSAtkins</td>
<td>RASOS</td>
<td>Lognormal COV 0.25</td>
<td>Lognormal COV 0.15</td>
<td>2.04E-04</td>
</tr>
<tr>
<td>Amoco</td>
<td>ASAS</td>
<td>Lognormal COV 0.34</td>
<td>unknown</td>
<td>9.30E-05</td>
</tr>
</tbody>
</table>

¹ - analysis performed as part of this research project

Table 49: Load and resistance parameters used in the comparison of minimal analyses

The difference between the Amoco prediction and the WSAtkins or the author’s predictions is likely to be due to a different representation of the resistance parameter. However, details relating to the resistance parameter were not available. In can be concluded that the results obtained in this research programme are in good agreement to those obtained by the independent WSAtkins study [WSAtkins, 1997b]. The 50-year extreme storm design base shear used in the SAFJAC minimal analysis approach and the RASOS system analysis approach, for the four wave approach directions studied are detailed in Table 50.

<table>
<thead>
<tr>
<th>Wave direction approach</th>
<th>SAFJAC design base shear¹ (MN)</th>
<th>RASOS design base shear (MN)</th>
<th>% difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Platform W</td>
<td>8.11</td>
<td>7.34</td>
<td>10%</td>
</tr>
<tr>
<td>Platform NW</td>
<td>7.97</td>
<td>7.19</td>
<td>10%</td>
</tr>
<tr>
<td>Platform N</td>
<td>6.04</td>
<td>4.12</td>
<td>38%</td>
</tr>
<tr>
<td>Platform SE</td>
<td>6.76</td>
<td>4.41</td>
<td>42%</td>
</tr>
</tbody>
</table>

¹ - analysis performed as part of this research

Table 50: 50-year design conditions used in SAFJAC and RASOS analyses
For non-linear foundations, the results derived by the author using SAFJAC and WSAtkins using RASOS, are summarised in Table 51 below:

<table>
<thead>
<tr>
<th>Wave direction approach</th>
<th>SAFJAC Ultimate base shear¹ (MN)</th>
<th>RASOS Ultimate base shear (MN)</th>
<th>% difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Platform W</td>
<td>14.05</td>
<td>14.67</td>
<td>4%</td>
</tr>
<tr>
<td>Platform NW</td>
<td>13.24</td>
<td>13.95</td>
<td>5%</td>
</tr>
<tr>
<td>Platform N</td>
<td>12.26</td>
<td>11.91</td>
<td>3%</td>
</tr>
<tr>
<td>Platform SE</td>
<td>12.85</td>
<td>15.57</td>
<td>19%</td>
</tr>
</tbody>
</table>

¹ - analysis performed as part of this research

Table 51: Ultimate base shear values derived from SAFJAC and RASOS analyses

The three analyses for waves approaching from platform W, NW and N show reasonable agreement of ultimate base shear between the methods, with differences ranging from 3% to 5%. However, for the wave approaching from the platform SE, the results differed by 19%, with the RASOS result being higher. The exact cause for this would need further information regarding the precise nature of the derivation of base shear and would require further detailed investigation that is outside the scope of this current research. It can be seen from Figure 69 that for the two approaches, the wave approach direction that corresponds to the highest ultimate base shear, is not in agreement. However, if the wave approaching from P-SE is considered spurious as implied above, then the author’s work using SAFJAC and the RASOS results show similar trends. The highest ultimate base shear is for the wave approaching from P-W, P-NW and P-N respectively.

Figure 69: Bar chart showing design and ultimate base shear results from SAFJAC¹ and RASOS analyses
Using the RASOS load model described earlier and representing the jacket resistance by a single variable, reliability calculations were carried out for each of the 8 wave approach directions. As part of this research, reliability calculations were performed for the 4 wave approach directions, using a single variable to represent the load and a single variable to represent the resistance. Amoco had also carried out a simplistic reliability analysis in 1996 (Stahl) using a model similar to WSAtkins, but instead of RASOS the software used was ASAS. This too was based on a two variable approach where both loading and resistance were each represented by a single variable. The loading COV used was 34.2% with a lognormal distribution. Reliability index results are included in the following table for comparison.

The results obtained from this research, compared with the results reported by WSAtkins using RASOS and Amoco using ASAS [WSAtkins, 1997b] are in Table 52 and Figure 70.

<table>
<thead>
<tr>
<th>Wave approach direction</th>
<th>Annual reliability index, $\beta$</th>
<th>Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Author$^1$</td>
<td>WSAtkins</td>
</tr>
<tr>
<td>Platform W</td>
<td>3.78</td>
<td>4.13</td>
</tr>
<tr>
<td>Platform NW</td>
<td>3.64</td>
<td>3.6</td>
</tr>
<tr>
<td>Platform N</td>
<td>5.11</td>
<td>4.31</td>
</tr>
<tr>
<td>Platform SE</td>
<td>4.54</td>
<td>5.2</td>
</tr>
</tbody>
</table>

$^1$ - analysis performed as part of this research

Table 52: Reliability analysis results, using lognormal distributions, using different methods, for four wave approach directions

When WSAtkins compared their results to the Amoco results, they found that “the two load models lead to reasonably similar estimates of the system reliability for the two critical directions (platform W and NW) while for other directions the results were somewhat different” [WSAtkins, 1997b]. No explanation for this was reported. Figure 70 highlights the fact that the results obtained in this research appear to place the different wave approach directions in a different sequence of criticality. The current results show that the waves are P-NW, P-W, P-SE and P-N (in order of decreasing criticality) whilst results from WSAtkins and Amoco show P-NW, P-W, P-N and P-SE.
It should be noted that the comparisons shown in Figure 69 and Figure 70 are not considered to be representative of the variation expected, but are specific to the structure studied, and the wave approach directions applied, for the three different independent approaches.

6.4 Application of the response surface technique in this research

6.4.1 Methodology

The response surface technique (RST) generates 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 resistance basic 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. It is recognised that the modelling uncertainty is an important issue, however it is beyond the scope of the current research to include it within the RST analysis work.

The central composite design (CCD) approach was used initially in this research. It can provide an efficient method for keeping the number of analysis points to a reasonably low level. The two main parameters to be considered by RST for this research were yield
strength, $\sigma_y$ and foundation capacity, $f_c$. Values for yield strength were derived assuming a lognormal distribution and a COV of 5%. These assumptions were based on previous studies into the effect of changing yield strength by Frieze/MSL [MSL Engineering, 1997], Imperial College [Chryssanthopoulos et al., 1986a; Chryssanthopoulos et al., 1986b; Chryssanthopoulos, 1992]. In each case a lognormal distribution was used to represent the variation in yield strength (see also Chapter 4).

When studying foundation capacity the single most important variable was considered to be the cone penetration test (CPT) end resistance as based on experience at Imperial College [Jardine and Chow, 1996b]. However, since it was not possible to extract this as a single parameter from the overall foundation model, an overall parameter, $f_c$, was designated to represent the foundation capacity. This was based on the ultimate capacity of the piles. A range of factors was applied in order to emulate changes to the foundation. Since no other information was available, it was decided to apply the factor to change the foundation capacity to both the axial and lateral capacities, as well as to the end bearing present in the compression case. With the lack of sufficient information on the expected distribution, a lognormal distribution was applied. Values for pile capacity were therefore derived assuming a lognormal distribution and a COV of 22% based on statistical analysis of available results used in previous studies by IC [Jardine and Chow, 1996b].

It was considered advantageous to select a soil profile that represented an initial mixed mode of failure in order to ascertain the role of yield strength and foundation capacity and their possible interaction. The ‘design profile’ (IC Case 1) was chosen as a suitable profile (see section 5.2 for details), which could be used to represent the centre point for the CCD response surface approach.

Results obtained from the parametric studies (see Chapter 4) into the effect of changing yield strength and the effect of changing the foundation capacity were studied. The results indicated that there was a predominantly linear relationship for the yield strength within the range examined, i.e. as the yield strength was increased then a proportional increase was exhibited in the peak load factor. Based on the ‘design profile’ the most appropriate region for further studies could be captured sufficiently with axial points at ±2 standard deviations, where the COV was 5%. In the case of the foundation capacity being changed, a different trend had been identified. As the foundation capacity was decreased and ‘softened’, a distinct decrease in the peak load factor was noted with a linear trend, which decreased as the foundation capacity was decreased. However, as the foundation capacity was increased,
after a point any increase in capacity did not show any increase in the peak load factor and
the overall trend appeared to plateau out. Using the IC ‘design profile’, the most interesting
and appropriate region for further studies was initially considered to be the area around the
change in trends. This was considered to be with axial points at ±2 standard deviations,
where the COV was 22%.

The ‘region of interest’ in this study was initially assumed to be around the mean. There
was no available information about the location of the ‘design point’ and it was therefore
reasonable to define the centre of the region of interest by the point and to consider an
experimental design built around it. In order to derive a suitable set of ‘experiments’ to
derive the response surface in an efficient way, using the CCD principle, the location of a
centre point, cube points and axial points were decided. Axial points are located at a
distance alpha (α) from the centre point [Chryssanthopoulos et al., 1986]. In this example,
according to the CCD principle, α^4 will equal the number of cube points (n_c), where α^4 = n_c.
Thus, if there are to be four cube points, α will be 1.414.

According to the CCD method, the axial points can be at some multiple of α if necessary.
Thus, for this example, the axial points could be located at 1.4α. If α was defined as being
equal to 1.414 standard deviations, then the position of the axial points would be at 1.4 x
(±1.4 standard deviations) which is ±2 standard deviations. Moreover, the runs for these
points had already been performed as part of the preliminary parametric studies. In order to
derive the cube points, n_c, these could be placed at points which are ±1.4 standard deviations
(σ) from the centre point. Nine observations were therefore performed for the response
surface technique. For each of these observations, a value for the peak load factor exhibited
in the pushover analysis was recorded. A three dimensional surface could therefore be
created using these nine points, in terms of the yield strength and foundation capacity,
against the values observed of the peak load factor. Figure 71 shows the observation points
used.

In order to represent the response surface, a number of different polynomial forms were
tried. For the nine observation points in the CCD approach it was found that a second order
polynomial could be used. However, closer examination of the accuracy of the surface
equation revealed that the difference between the surface prediction and the FE observations
increased with decreasing foundation capacity. It was also concluded that the initial region
adopted for the foundation capacity (of ±2 standard deviations) was insufficient, and did not
provide enough data, particularly at the lower foundations capacities. The range was
therefore extended to ±2.8 standard deviations. It was important to fully capture the shape of the surface particularly at low foundation capacities. Additional observations were therefore performed, in terms of both additional axial points as well as extra cube points (as shown in Table 53). It was found that a higher order polynomial was then required to accurately represent the surface. Additional points at higher foundation capacities were also introduced in order to capture fully the tail end distributions, and to ensure that the polynomial fit was appropriate for all cases across the range to be expected.

![Figure 71: Location of the nine runs performed for the CCD](image)

### 6.4.2 Summary of intermediate results

A total of 29 observation points was therefore used to fit the polynomial expression for the response surface. This number of points was required in order to ensure that the polynomial fitted to the data gave a good representation of the trends at both low and high foundation capacities. A way in which the number of points could be reduced, could be to represent the surface by a bi-polynomial function, rather than as one continuous function. However, for the present study a 5th order polynomial was found to give an adequate fit. Lower order polynomials gave poor fit to the data at the lower and higher foundation capacities. This equation was in the following form:

\[
R = a + b(\sigma_y) + c(f_c) + d(f_c^2) + e(\sigma_y, f_c) + f(f_c^3) + g(f_c^4) + h(f_c^5)
\]

Where: \(R\) = response, \(\sigma_y\) = yield strength, \(f_c\) = foundation capacity.

Values for the coefficients \(a\) to \(h\) were derived using an iterative technique to minimise the sum of the error squared and was performed using Solver in Microsoft Excel.

\[
\begin{align*}
a &= -4.9822, \\
b &= 0.00009, \\
c &= 1.7550, \\
d &= -0.1646, \\
e &= 0.000001, \\
f &= 0.007249, \\
g &= -0.0001510, \\
h &= 0.0000012
\end{align*}
\]
Table 53 shows the locations of each observation, in terms of standard deviation of the yield strength and foundation capacity, along with the pushover analysis peak load factor (LF) exhibited. The surface predicted peak load factor is then shown, and compared to the FE measured value derived as a bias indication. The percentage difference between the FE and surface predicted peak load values are shown in the final column.

<table>
<thead>
<tr>
<th>Yield strength (in standard deviations)</th>
<th>Foundation capacity (in standard deviations)</th>
<th>Pushover analysis peak load factor</th>
<th>Predicted peak load factor</th>
<th>Peak LF</th>
<th>Peak LF&lt;sub&gt;meas&lt;/sub&gt;</th>
<th>% difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.1</td>
<td>-2.8</td>
<td>0.473</td>
<td>0.515</td>
<td>1.08</td>
<td>1.07</td>
<td>8%</td>
</tr>
<tr>
<td>-0.9</td>
<td>-2.8</td>
<td>0.473</td>
<td>0.514</td>
<td>1.07</td>
<td>1.07</td>
<td>7%</td>
</tr>
<tr>
<td>-0.4</td>
<td>-2.3</td>
<td>1.040</td>
<td>1.143</td>
<td>1.09</td>
<td>1.07</td>
<td>8%</td>
</tr>
<tr>
<td>-2.8</td>
<td>-2.0</td>
<td>1.435</td>
<td>1.457</td>
<td>1.02</td>
<td>1.02</td>
<td>2%</td>
</tr>
<tr>
<td>2.0</td>
<td>-2.0</td>
<td>1.435</td>
<td>1.463</td>
<td>1.02</td>
<td>1.02</td>
<td>2%</td>
</tr>
<tr>
<td>mean</td>
<td>mean</td>
<td>1.435</td>
<td>1.462</td>
<td>1.02</td>
<td>1.02</td>
<td>2%</td>
</tr>
<tr>
<td>0.1</td>
<td>-1.8</td>
<td>1.671</td>
<td>1.635</td>
<td>0.98</td>
<td>-2%</td>
<td></td>
</tr>
<tr>
<td>-0.9</td>
<td>-1.8</td>
<td>1.671</td>
<td>1.634</td>
<td>0.98</td>
<td>-2%</td>
<td></td>
</tr>
<tr>
<td>-0.4</td>
<td>-1.6</td>
<td>1.765</td>
<td>1.729</td>
<td>0.98</td>
<td>-2%</td>
<td></td>
</tr>
<tr>
<td>-1.4</td>
<td>-1.4</td>
<td>1.906</td>
<td>1.842</td>
<td>0.97</td>
<td>-3%</td>
<td></td>
</tr>
<tr>
<td>1.4</td>
<td>-1.4</td>
<td>1.911</td>
<td>1.846</td>
<td>0.97</td>
<td>-3%</td>
<td></td>
</tr>
<tr>
<td>-0.6</td>
<td>0.4</td>
<td>1.985</td>
<td>2.085</td>
<td>1.05</td>
<td>5%</td>
<td></td>
</tr>
<tr>
<td>1.4</td>
<td>1.4</td>
<td>2.085</td>
<td>2.018</td>
<td>0.97</td>
<td>-3%</td>
<td></td>
</tr>
<tr>
<td>-1.4</td>
<td>1.4</td>
<td>1.954</td>
<td>2.015</td>
<td>1.03</td>
<td>3%</td>
<td></td>
</tr>
<tr>
<td>mean</td>
<td>mean</td>
<td>2.006</td>
<td>2.098</td>
<td>1.05</td>
<td>5%</td>
<td></td>
</tr>
<tr>
<td>-2.8</td>
<td>mean</td>
<td>1.893</td>
<td>2.095</td>
<td>1.11</td>
<td>10%</td>
<td></td>
</tr>
<tr>
<td>2.0</td>
<td>mean</td>
<td>2.080</td>
<td>2.101</td>
<td>1.01</td>
<td>1%</td>
<td></td>
</tr>
<tr>
<td>mean</td>
<td>2.0</td>
<td>2.028</td>
<td>1.983</td>
<td>0.98</td>
<td>-2%</td>
<td></td>
</tr>
<tr>
<td>2.0</td>
<td>2.0</td>
<td>2.148</td>
<td>1.986</td>
<td>0.92</td>
<td>-8%</td>
<td></td>
</tr>
<tr>
<td>-2.8</td>
<td>2.0</td>
<td>1.925</td>
<td>1.979</td>
<td>1.03</td>
<td>3%</td>
<td></td>
</tr>
<tr>
<td>mean</td>
<td>2.6</td>
<td>2.006*</td>
<td>1.969</td>
<td>0.98</td>
<td>-2%</td>
<td></td>
</tr>
<tr>
<td>mean</td>
<td>3.5</td>
<td>2.006*</td>
<td>1.983</td>
<td>0.99</td>
<td>-1%</td>
<td></td>
</tr>
<tr>
<td>mean</td>
<td>4.4</td>
<td>2.006*</td>
<td>2.029</td>
<td>1.01</td>
<td>1%</td>
<td></td>
</tr>
<tr>
<td>mean</td>
<td>5.2</td>
<td>2.006*</td>
<td>2.077</td>
<td>1.04</td>
<td>3%</td>
<td></td>
</tr>
<tr>
<td>mean</td>
<td>6.1</td>
<td>2.006*</td>
<td>2.097</td>
<td>1.05</td>
<td>4%</td>
<td></td>
</tr>
<tr>
<td>mean</td>
<td>7.0</td>
<td>2.006*</td>
<td>2.073</td>
<td>1.03</td>
<td>3%</td>
<td></td>
</tr>
<tr>
<td>mean</td>
<td>7.9</td>
<td>2.006*</td>
<td>2.015</td>
<td>1.00</td>
<td>0%</td>
<td></td>
</tr>
<tr>
<td>mean</td>
<td>8.8</td>
<td>2.006*</td>
<td>1.972</td>
<td>0.98</td>
<td>-2%</td>
<td></td>
</tr>
<tr>
<td>mean</td>
<td>9.7</td>
<td>2.006*</td>
<td>2.054</td>
<td>1.02</td>
<td>2%</td>
<td></td>
</tr>
</tbody>
</table>

Table 53: Peak load factor results and predictions from 29 observation points
(* values assumed based on previous observations)

Table 53 shows the results obtained for the peak load factor from the pushover analyses performed using SAFJAC and compares them with the predicted value derived from the equation that represents the response surface. It can be seen that the equation fit varies slightly across the points and that the maximum difference was 10%. This was for the axial point at -2.8 standard deviations for the yield strength and at a mean foundation capacity.
It was necessary to ascertain whether the equation derived for the surface adequately represented the data and to ensure that no other trends had been overlooked. To check this graphs were produced showing the comparison of the bias of the predicted and measured peak load factor against both yield strength and foundation capacity. If no trend was detected then the equation for the surface was assumed to be adequately representing the data. Figure 72 shows the results obtained against yield strength and Figure 73 show results against foundation pile capacity. It can be seen that there is no trend or skewing of the data.

Figure 72: Predicted peak load factor divided by measured peak load factor against yield strength of the deck columns, jacket legs and braces

Figure 73: Predicted peak load factor divided by measured peak load factor against foundation pile capacity
6.4.3 Derivation of reliability index and probability of failure

Once the checks had been performed to ascertain that the equation describing the response surface was acceptable, the equation was input into the probabilistic analysis program, PROBAN. A distribution for both the yield strength and the foundation pile capacity were applied. The loading was described in PROBAN based on a lognormal distribution with a COV of 20%. The limit state, $G$, was derived by taking into account the resistance, $R$ and the loading, $L$ and was defined as $G = R - L$. The reliability index and probability of failure were derived using both the first order reliability method (FORM) and the second order reliability method (SORM).

The estimation for the probability is the point on the limit state surface that is closest to the origin and this gives the most likely failure point in the standard normal space. Information was obtained about the location of the design point from PROBAN which was described by values for the load, yield strength and foundation capacity.

Table 54 and Table 55 show the set up and results derived using PROBAN where the lower bound for was zero.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Mean $\mu$, COV and standard deviation $\sigma$ values</th>
<th>Distribution characteristics</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load</td>
<td>$\mu = 0.561$MN, COV = 0.2, $\sigma = 0.1122$</td>
<td>Lognormal (lower bound = 0)</td>
</tr>
<tr>
<td>Yield strength</td>
<td>$\mu = 250$ MPa, COV = 0.05, $\sigma = 12.5$MPa</td>
<td>Lognormal (lower bound = 0)</td>
</tr>
<tr>
<td>Foundation capacity</td>
<td>$\mu = 12.81$ MN, COV = 0.22, $\sigma = 2.8$ MN</td>
<td>Lognormal (lower bound = 0)</td>
</tr>
</tbody>
</table>

Table 54: Set up used in the reliability analysis in PROBAN

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reliability index, $\beta$</td>
<td>FORM $= 4.120$</td>
</tr>
<tr>
<td></td>
<td>SORM $= 4.097$</td>
</tr>
<tr>
<td>Probability of failure, $P_f$</td>
<td>FORM $= 1.893E-05$</td>
</tr>
<tr>
<td></td>
<td>SORM $= 2.093E-05$</td>
</tr>
<tr>
<td>Load design point location</td>
<td>0.649</td>
</tr>
<tr>
<td></td>
<td>(mean $= 0.561$)</td>
</tr>
<tr>
<td>Yield strength design point location</td>
<td>249.6 MPa (mean $= 250$)</td>
</tr>
<tr>
<td>Foundation capacity design point location</td>
<td>5.235 MN (located at $-2.7\sigma$)</td>
</tr>
</tbody>
</table>

Table 55: Results obtained from PROBAN for the design point from the response surface analysis (using 5th order polynomial equation)
In order to verify whether the surface was an accurate representation of the response, it was necessary to compare the predicted peak load factor derived at the design point with measured results. The measured result at the design point was derived by performing one additional pushover analysis using the yield strength and foundation capacity values as predicted by the reliability analysis. If the prediction of peak load at this point was within a suitable tolerance, then the expression describing the failure surface could be approved and hence the reliability results derived would be considered acceptable. Previous studies have shown that if the prediction at the design point was within 5%, the results could be considered acceptable [Chryssanthopoulos et al., 1986; Chryssanthopoulos et al., 1987]. If the prediction is more than say 5% different, then a new set of runs would need to be performed around the design point and the analysis repeated until the results were within the required tolerance. The results from the pushover analysis performed at the design point and the prediction analysis are shown in Table 56.

<table>
<thead>
<tr>
<th>Analysis</th>
<th>Peak load factor</th>
<th>Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prediction from response surface</td>
<td>0.647</td>
<td>+5.23%</td>
</tr>
<tr>
<td>Measured from pushover analysis</td>
<td>0.614</td>
<td></td>
</tr>
</tbody>
</table>

Table 56: Difference in prediction and measured peak load factor at the design point

It can be seen that the difference between the measured and predicted peak load factors is 5.23%, with the surface over-predicting the peak load factor measured from the pushover analysis. This only marginally exceeds the nominal 5% tolerance used by others and it was decided that the surface equation would be taken as a valid representation of the response surface. This means that the reliability results can be taken to be a reasonable representation of the reliability for the structure. Modelling uncertainty will also exist but in this research its effect has not been incorporated into the response surface. Further work would be needed in order to assess a suitable level of this effect as well as the method in which it should be applied.

6.4.4 Other relevant investigations

A study undertaken by MSL Engineering to investigate the reliability of jacket and jack-up structures made use of the response surface technique [MSL Engineering, 1997]. The jacket structure used in this study was based on Shell’s Kittiwake structure located in the central North Sea. The soil profile adopted was clay over sand over clay. Two scenarios were studied: one with a jacket only failure mode, where foundation failure was prevented by redesigning the piles with a small diameter to wall thickness ratio and the other with foundation failure occurring before any structural members failed. Response surfaces were
developed for the foundation failure case, relating system strength to the pile capacity \( P \), for two wave approach directions from the North and North West. The response surfaces derived were of the general form: \( R = a \times P - b \) where \( a \) and \( b \) were different constants for the two wave approach directions. It should be noted that it was not possible to obtain information regarding the precise details of the basis of the analysis work, including values used to derive the surface, or details of the assessment of the design point, and therefore its significance is clearly limited.

6.5  Report on third party application of the system analysis approach

Rigorous “system” reliability analysis requires substantial computational effort, and research work in recent years has concentrated on the development of efficient methods for identifying the most dominant failure paths and deriving the combined system probability of failure. An example of this approach is that used by WSAtkins who undertook a reliability assessment of Leman AP in February 1997 [WSAtkins, 1997b]. The objective was to assess the effect of damage on the integrity of the jacket for different damage scenarios. The software RASOS was used for both the non-linear collapse and the system reliability analyses.

6.5.1 Methodology

Preliminary deterministic analyses were used to calculate the non-linear response of the jacket for eight wave approach directions under the 50-year return loading conditions. First member failure, ultimate collapse and deterministic redundancy were derived for each direction. System reliability analysis based on a failure-tree approach was performed for the direction identified as critical. A simplified method was used to estimate the system reliability for other directions, where a single random variable was used to represent resistance variability. This required the development of probabilistic models for the annual maximum base shear due to environmental loading and the ultimate strength.

Piled foundations were modelled by a combination of non-linear pile elements and linear support springs, representing soil-pile interaction. Pile springs in the three (translation) degrees of freedom, were distributed along the pile length at 10 locations. These were calibrated based on the deflection response at the pile cap of a single pile under the action of a “near collapse” load. Springs were assumed the same for each wave direction. Separate beam or column elements were used for piles and legs, with constraints introduced at leg node levels to ensure identical displacement of legs and pile in the transverse direction. In the axial direction, legs and piles were free to move.
The applied loading consisted of 5 types: self weight of the jacket and appurtenances loading, weight of deck equipment, wind loading on the deck, buoyancy loading and combined wave and current loading on the jacket. The self-weight of the jacket and appurtenances loading and weight of deck equipment were applied as nodal forces, moments and distributed member loading. The wind loading was applied as forces and moments applied at the connecting nodes at the deck level of the jacket. Environmental loading used the 50-year extreme storm condition, with the corresponding sea state parameters for the 8 wave approach directions. Progressive collapse analysis was performed using an incremental approach that allowed for the failure of all components including pile, members and joints. It was found that the dominant failure mode included all types of components.

In the system reliability analysis, a failure tree approach was adopted. Stochastic models for the two basic variables loading and resistance were developed. The selective enumeration technique was applied in order to identify the dominant failure modes of the structure. Subsequent analysis enabled the upper and lower bounds of the jacket probability of failure to be derived. The extensive analysis was undertaken for one wave direction only – that from platform West. For the reliability analysis, six piles below the mudline along with 50 tubular members were assumed to have random resistance properties, whilst the remaining members were considered to have a deterministic resistance.

### 6.5.2 Summary of intermediate results

The following table shows the results based on the four wave-approach directions studied by WSAAtkins using the system analysis approach [WSAAtkins, 1997b]. Table 57 shows the 50-year design base shear values and the ultimate base shear results.

<table>
<thead>
<tr>
<th>Wave direction approach</th>
<th>50 yr. design base shear (MN)</th>
<th>Ultimate base shear (MN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Platform W</td>
<td>7.34</td>
<td>14.67</td>
</tr>
<tr>
<td>Platform NW</td>
<td>7.19</td>
<td>13.95</td>
</tr>
<tr>
<td>Platform N</td>
<td>4.12</td>
<td>11.91</td>
</tr>
<tr>
<td>Platform SE</td>
<td>4.41</td>
<td>15.57</td>
</tr>
</tbody>
</table>

Table 57: Design and ultimate base shear results from the system analysis approach

### 6.5.3 Derivation of reliability index and probability of failure

WSAAtkins assumed the uncertain parameters of the RASOS load model used for the Leman AP jacket included wave height, current speed, marine growth and load model bias. In the analysis, the mean values of wave height and current velocity were taken to be the annual maximum values, as estimated from the design environmental data given for the 50-year return period. Table 58 shows the models that were used for the load parameters:
Table 58: Stochastic model for basic load parameters used by WSAtkins [WSAtkins, 1997b]

The COV of the yield strength was estimated to be approximately 8%, but in order to account for geometric uncertainties, a COV of 12% was used to represent variability of capacity of tubular members. The COV of pile capacities was assumed to be 25%. Thus, the following models were used for the resistance parameters in the system analysis reliability approach as shown in Table 59:

Table 59: Stochastic model for basic resistance parameters used in RASOS analyses

The reliability results were for the analysis performed for the wave approaching from platform West. The reliability index and probability of failure were derived for the member most likely to fail first, the first failure of any member, the most likely complete failure path and the system failure through any path. Table 60 shows the system reliability analysis results for the intact structure of Leman AP with foundations included in the model.

Table 60: Reliability results for Leman AP for wave from platform West [WSAtkins, 1997b]

To date, no other studies have been performed using the system analysis method on the Leman AP platform and it was therefore not possible to compare the results obtained.

6.6 Overall comparison of reliability analysis methods

6.6.1 Loading

In order to predict lifetime extremes of environmental parameters, the available data are extrapolated and therefore large uncertainty can be expected. In the minimal
analysis approach as applied in the cases considered in this study, a simple distribution is used for the loading defined with a mean value and a COV. Instead of representing each of the individual loading parameters, a global distribution representing the overall loading is applied. The statistics of this variable are usually determined from that of the wave height alone, based on the assumption that a simple relationship exists between the wave height and the member force. As this approach does not take into account the uncertainties in the individual environmental parameters, such as would be the case with NewWave from Shell [Tromans et al., 1993; van de Graaf et al., 1993], it gives an estimate in modelling the variability in environmental conditions, which can adversely affect the reliability results. A similar approach of using a simple overall distribution for the loading is often used in conjunction with the response surface technique, although more detailed loading models can be combined with both the minimal and response surface methods. In the approach used for this research, for this method, the focus is on the resistance side rather than the loading.

In contrast, in the system analysis approach examined above, individual components are combined to make up the loading model. Uncertainties in the individual parameters are accounted for by individual distributions and COVs. In general, only the key parameters are represented as variables, whilst those considered as less significant will be treated as deterministic. Thus, random variables of low uncertainty or low sensitivity will be replaced with deterministic equivalents. Statistical techniques are therefore used to assess the metocean data and then to develop a probabilistic model for the extreme environmental condition.

Irrespective of the method chosen above, it is important to note that a joint probability distribution of all the environmental parameters is required, as they are generally seen to be highly correlated. In practical situations, enough data are rarely available to develop this joint distribution. To move towards more consistent reliability analysis, this would be the preferred option, as it more accurately represents the environmental loading.

Solely within the context of the methods as applied and studied in this research, the differences in dealing with the environmental loading of the three methods described above have shown that two key approaches are generally adopted. These approaches are quite different and place different emphasis on the need to fully represent the individual parameters of the environmental loading model. In order to move towards more consistent reliability the best possible representation of the environmental loading is required. For the methods applied in this study, for the loading, this would therefore be through the use of the
technique described within the system analysis approach, in terms of representing the individual parameters that make up the environmental model. However, one disadvantage of this approach is that it requires significantly more data and requires notably more analysis work in order to derive the appropriate distribution and COVs for each parameter. Such data may not be available and the analysis may thus be forced to use the simple distribution approach. However, if a wider perspective is taken, it should be noted that these simple distributions of loading have been based up experience of the ‘best estimate’ of load and its distribution. There are other techniques developed, such as that developed by Shell as NewWave [Shell Research, 1993] where a rigorous and detailed processing of the raw environmental data is developed into one accurate but ‘simple’ distribution of the loading. In such a case, the resulting ‘simple’ loading representation has been derived from a more elaborate, complex and detailed approach. Either of these loading representations can be used in conjunction with the minimal analysis approach or the response surface technique.

6.6.2 Resistance

It has been shown that the method in which the resistance or response of the structure is represented can differ significantly according to which approach is used. The probabilistic description for the strength and stiffness of structural members depends upon the probabilistic description of the members and joints, such as the cross-sectional dimensions and material strengths. In the minimal analysis approach, one simple distribution is adopted to represent the structure’s overall resistance to environmental loading. In doing so, it does not take into account the uncertainty in the individual parameters that combine to form the resistance.

In the response surface technique, the global response of the structure is represented not by a distribution, but by a surface. Provided that this is calibrated correctly, it can be constructed out of any number of the individual response parameters. The example detailed above has focused on the foundation capacity and the yield strength parameters. The response surface that is derived is only valid for an individual wave direction and, in addition, will only be applicable to the structure under consideration. Other wave approach directions are likely to have different structural configurations and hence different responses. The RST method provides a thorough approach to representing the resistance of a structure. However, difficulties can arise if the design point predicted lies outside the area of the surface derived. In this case, additional analyses will be required in order to represent the response more accurately. In addition, it is vital that the form of the equation adopted accurately represents
the response over the full range of the individual parameters. It is important that it does not produce misleading results when examined outside the region in which it was fitted. In the system analysis approach, the resistance is derived from the ultimate capacity of the structure. The piles and a selection of tubular members are usually assumed to have random resistance properties, while the capacities of remaining members are usually taken as deterministic. The members are usually selected because of high utilisation ratios at some stage during the deterministic collapse analyses.

Of the three methods studied, the approach adopted in the response surface technique represented the most rigorous representation of foundation reliability effects. Within the system approach, the reliability of foundations was represented by the introduction of a selected number of pile elements with random resistance properties. In the minimal analysis method, no attempt was made to incorporate foundation reliability effects.

The differences in dealing with the resistance in the three methods described above have shown that the resistance can be treated with varying degrees of complexity. In order to move towards more consistent and 'true' reliability the best possible representation of both the environmental loading and the resistance is required. Careful application of RST, which can make use of general-purpose non-linear finite element and reliability software, can provide a good practical approach.

6.6.3 Reliability

The reliability of a structure is based on the possibility of the resistance being inadequate for the acting load or that the load exceeds the available strength. In the minimal analysis approach, the reliability is simply derived from the two simple representations of strength and loading. A failure surface can then be constructed from the results derived for different wave approach directions, on a compass diagram. If an overall representation of reliability is required then the probability of failure derived for the two 'worst' wave approach directions can be combined, as discussed previously:

$$\text{Overall } P(f) = P(f)_1 + P(f)_2 - P(f)_1 \cdot P(f)_2$$

Where $P(f)_1$ and $P(f)_2$ are the probability of failure for different wave approach directions.

The reliability results from the response surface approach can be obtained by inputting the loading distribution, along with an equation and distributions for the resistance. This can be performed in software such as PROBAN. The results are in terms of both the reliability index and probability of failure, along with details as to the design point location. The location of the design point in terms of the input parameters enables the accuracy of the
surface to be assessed. However, due to the complex interaction of the loading distribution with a response surface, it is not possible to perform a manual check on the results. In addition, the surface may be sensitive to changes in the distribution of response parameters. If care is not taken, an incorrect distribution adopted may cause inappropriate results.

In the system analysis approach, the reliability analysis is based upon a failure sequence approach, also called a cut-set. In this case, a parallel system represents the structure where every component in the sequence must fail in order that the structure will fail in a selected failure mode. However, for an offshore structure, the number of possible failure mechanisms is large and structural failure may result from any one of these modes. Since it would be impractical to identify all of the failure paths, a number of ‘dominant’ failure paths are selected for subsequent use in the system reliability analysis. Due to the various approximations required, this kind of method does not guarantee that none of the dominant failure modes will be overlooked. It is important to perform such analysis with previous knowledge obtained from a simplified or deterministic analysis. The ‘selective enumeration method’ was developed to reduce the computational effort of considering all surviving elements [Shetty, 1994]. The method has been successfully applied to the system reliability analysis of a number of jacket structures, and is an efficient search algorithm.

### 6.6.4 Comparison of results

In the case of the Leman AP structure, the following results can be compared. Using the minimal analysis approach on Leman AP with a wave approaching from Platform North, a reliability index of 5.11 was derived. An overall probability of failure of 2.14E-04 was obtained. These were based on lognormal loading with a COV of 0.2 and lognormal resistance with a COV of 0.15. Using the response surface approach on Leman AP with a wave approaching from Platform North, a reliability index of 4.120 was derived, with a probability of failure of 1.89E-05. This was based on lognormal loading with a COV of 0.2 and a response surface derived from lognormal yield strength with COV of 0.05 and lognormal foundation capacity with a COV of 0.22.

However, using the system reliability approach on Leman AP with a wave approaching from Platform West, results obtained by WSAtkins gave a lower bound and an upper bound of the reliability index and the probability of failure. For the first failure of any member, the reliability index was 3.23 with a probability of failure of 6.19E-04. For the most likely complete failure path, the reliability index derived was 4.48 with a probability of failure of 3.74E-06. These key results are shown in Table 61.
Table 61: Key reliability results for Leman AP derived by different methods

Although these reliability index and probability of failure results are derived from significantly different approaches, it is interesting to note that the maximum difference between the highest and lowest predicted reliability indices is 13%. It can also be seen that the reliability index is lowest for the response surface analysis and highest for the minimal analysis technique. Care must be taken when comparing the results that have been derived for different wave approach directions and by different methods. Consequently, the results presented in Table 61 should be compared with caution. It should be noted that the comparisons are not considered to be representative of the variation expected, but are specific to the structure studied, and the wave approach directions applied, for the three different independent approaches.

6.7 Preliminary development of a simplified system analysis approach

While the majority of structural codes in the UK specify that structures are designed on a member-by-member basis, most elements within a structure are actually performing as part of a complicated structural system. Interest in characterising the performance and safety of structural systems has led to an increased interest in the area of system reliability. The classical theories of series and parallel system reliability are well developed and have been applied to the analysis of such complicated structural systems as offshore structures.

6.7.1 Simplified system analysis methods in published works

6.7.1.1 Cornell

In 1994, Cornell worked on the development of a random-variable level probabilistic model of structural demand, behaviour and capacity [Cornell, 1995]. This work was based on near-failure, static/dynamic displacement behaviour of structural systems using an explicit analytical form [see also Cornell, 1994]. Cornell noted from extended research that it was sufficient to assume with respect to any particular extreme load indicator, that the failure probability of a system with multiple modes of failure was approximately equal to the maximum of the individual modal failure probabilities. Furthermore, if a system had
redundancy, the net reliability with respect to the loads was marginally better than that of
the strongest path. It was concluded that the main reason for this was the relatively large
random variability in the extreme load and relatively large variability of the capacities of the
modes of failure [Cornell, 1995].

Cornell also concluded that an adequate rule for systems was to check all potential failure
modes. If the weakest or maximum probability of failure mode satisfies the safety criterion,
then the system as a whole does. Furthermore, the mode and hence the system are usually
not much better than the strongest element, so little potential system reliability benefit is
being ignored by this simple rule [Cornell, 1995]. The main point of this rule being that if a
deterministic capacity analysis is performed correctly, there is little gain or loss compared to
considering the system capacity probabilistically. However, there are notable exceptions to
this simple rule - these being fatigue and robustness with respect to local flaws and localised
impacts. The author did not specifically mention aspects relating to correlation.

Cornell concluded that for the case of a jacket subjected to extreme weather hazard (waves,
wind and current) it was sufficient to assume that the reliability of the structure was that of
its most likely failure mode, which in most cases, is effectively its deterministically weakest
mode of failure [Cornell, 1995]. It was also noted that it is important to estimate the
ultimate capacity of the structure and that detailed, precise analysis far from the “near-
failure” condition was not as beneficial as a less detailed, rougher analysis of the “near
failure” condition. The burden should be on the analyst to try to predict failure not to over-
study a conservative design basis condition set at a probability of exceedence level well
above the probability of failure level.

Cornell also investigated the inclusion of uncertainties in reliability based design and
concluded that it was good professional practice to report the probabilistic and physical
uncertainties. It was concluded that the most direct way to include them in practice was to
simply use the mean probability, i.e. the average over these uncertainties, or the so-called
“predictive probability”. However, it was noted that there was a need for much more
experience with the assessment and application of such uncertainties.

6.1.3.1 Bea

Bea developed several approaches for evaluating the acceptable, tolerable or desirable
reliability of a structure [Bea, 1991; Bea, 1993a; Mortazavi and Bea, 1996; and Bea et al.,
1997]. The most recent approach developed by Bea et al [Bea et al., 1997], was applied to
the reassessment and re-qualification of two Gulf of Mexico platforms. The analysis
procedure consisted of 3 levels as developed in the API guidelines for reassessment and requalification of steel template-type offshore platforms: screening analysis, design level analysis (DLA) and ultimate strength analysis (USA). The three levels of analysis were to be performed sequentially, with the checks becoming more detailed and less conservative.

Bea et al reported on two types of analysis - the design level analysis using the program StruCAD*3D and the ultimate strength analysis using the programme ULSEA developed in 1995 by Bea.

In the DLA approach, structure loading and capacity were calculated using the API RP2A-WSD (1993) and soil structure interaction was evaluated from pile geometry and soil characteristics. The DLA capacity of a member was determined by the creation of the first plastic hinge or member yielding. In the ultimate strength approach, the platform's lateral loading capacity was determined by using plastic hinge theory, using specialised software named ULSEA - after the ultimate limit state equilibrium analysis techniques developed. The structure was divided into three main components: deck legs, jacket and pile foundation as shown in Figure 74:

![Diagram showing method by which Bea subdivided the platform structure](image)

Figure 74: Diagram showing method by which Bea subdivided the platform structure

For the deck legs, the mode of failure was defined as plastic hinge formation and subsequent collapse of the deck portal. For the jacket, failure was defined as buckling of primary vertical diagonal braces within the jacket. For the pile foundation, failure was defined as the formation of plastic hinges in the piles, resulting in lateral failure, or by axial pile failure due to pullout of plugging.

Bea developed the program ULSEA in 1995 as the basis for his ultimate strength analysis (USA). Within this, aerodynamic and hydrodynamic loadings were calculated using API RP2A-WSD 1993. The wind speeds at +30ft, wave height, wave period and current profile were all user-defined. Water particle velocities and accelerations were determined using Stokes V wave theory. Morison's equation was used to estimate hydrodynamic loading with directional spreading, current blockage factors, inertia, drag and marine growth being user-defined. Structural elements, including appurtenances, were modelled as equivalent vertical cylinders that were located at the wave crest and for inclined members, the effective projected vertical area was used.
Plastic hinge theory was used to define the platform lateral loading capacity and the ultimate shear capacity of the deck portal was estimated based on the bending moment capacities of the deck legs. The jacket structure was subdivided into its bays and the capacity of each bay determined separately. The vertical bracing strength at each bay was estimated as the minimum capacity of the brace or the capacity of either of its joints. A lower and an upper bound capacity were defined for each bay: the lower bound was the capacity of the member most likely to fail i.e. first member failure and the upper bound was the residual strengths of all braces within that bay. Brace capacity was defined as yield when in tension and buckling when in compression and joint capacity was defined as the first crack that develops in tension and in compression.

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 was when the storm shear just exceeded the platform's horizontal capacity. A reliability study was then 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 sea states 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.

6.7.2 Background to ideas for new initial screening tool

In order to obtain an estimate of the strength of a platform and the critical failure mode to be expected, a pushover analysis will generally be performed. As described in previous sections foundations in the past were excluded from the analysis based on inspection and observation and engineering judgement and were not always modelled in detail [Gierlinski et al., 1993; Sigurdsson et al., 1994; Tromans et al., 1993 and Light et al., 1995]. This has since been found potentially conservative for piles in sands and unconservative for some clay foundations [Jardine and Chow, 1996b and MSL Engineering, 1997]. In order to make adequate assessments and predictions regarding structures with the possibility of foundation failure, the foundations must be modelled in detail. Theoretically, this could be performed by a detailed 3D non-linear FE modelling of the foundations. However, this is not always practical given the size of combined platform and foundation computer model. A more realistic representation is to model the foundations using non-linear beam elements representing the piles and non-linear springs representing the soil stiffness [BOMEL, 1992].
An extensive series of pushover analyses have been performed within this research programme (see Chapters 4 and 5) and it was found that there were three main possible outcomes in terms of failure modes. These were jacket only failure, foundation only failure and mixed mode failure. Failure modes such as fatigue, pile pullout and failure associated with large deck displacements that would exceed equipment limitations were not included. As described in Chapter 4, for all failure modes the initial load-deflection characteristics are very similar. However, for the failure mode dominated by foundation failure, there is a gradual softening of the response. As the load is increased, an ever-increasing deflection is exhibited, until a point is reached when only a very small increase in the load leads to very large deflections. Conversely, for the failure mode dominated by jacket failure, the system exhibits a stiffer response and a distinct peak is exhibited on the load-deflection characteristics. Post-peak there is slight unloading of the system, before a gradual increase is exhibited again. Three ‘typical’ pushover analysis cases are shown in Figure 75.

![Load deflection characteristics for three pushover analysis cases studied with different failure modes](image)

Figure 75: Load deflection characteristics for three pushover analysis cases studied with different failure modes

For the failure mode that consists of a combination of jacket failure and foundation failure, i.e. a mixed mode, the load-deflection characteristics are not as well defined as for the foundation only case or the jacket only case. The mixed mode load-deflection characteristic is represented by some combination of those for the two individual failure modes. As described in Chapter 4, three distinct regions can be identified on the curve constructed from peak load factor exhibited against foundation capacity, as seen in Figure 76. It can be concluded that for those analyses where foundation failure was dominant the effect of yield strength in the structure was minimal. For those analyses where jacket failure dominated,
then yield strength is the dominant factor and the effect of foundation capacity in the structure was negligible. For those analyses where a mixed mode failure was exhibited, both yield strength and foundation capacity would have to be taken into consideration.

![Diagram](image)

**Figure 76: Trendline curve of peak load factor against foundation capacity**

This theory can be further developed in order to enable assessments to be made of what the dominant failure mode is, within a certain range, for a jacket type structure. If the structure exhibited a failure clearly in the jacket-dominated region, then detailed foundation assessments would be superfluous. If however, the structure exhibited failure clearly in the foundation dominated region, then detailed yield strength assessment would not be necessary. If the structure exhibited a mixed mode failure, then both the foundation capacity and the yield strength would need to be studied in detail.

### 6.7.3 Proposed basis of assessment

In order to make a reasonable assessment of the failure scenario it is important to establish what is the required range of foundation capacity that would need to be studied. It was proposed that for the structure considered a range of ±n standard deviations from the mean would be sufficient to capture the failure scenario. If a near-horizontal linear relationship were established from the three data points of mean, +n standard deviations, and −n standard deviations, then this would indicate a jacket dominated failure scenario. If a positive linear relationship were established, then this would indicate a foundation dominated failure scenario. If however, a linear fit were inappropriate for the data set, then this would indicate a mixed mode failure respectively. These three scenarios are shown in the diagrammatic representations in Figure 77, Figure 78 and Figure 79 for jacket-dominated, foundation-dominated and mixed mode failures.
Figure 77: Foundation capacity against peak load factor for *jacket-dominated* failure

Figure 78: Foundation capacity against peak load factor for *foundation-dominated* failure

Figure 79: Foundation capacity against peak load factor for *mixed-mode* failure scenario
For the foundations dominated and jacket dominated failure modes identified, over a certain range of ±\( n \) standard deviations, it was hypothesised that it might be possible to represent the response surface with a linear approximation without significant loss of accuracy. For the foundations dominated range, this linear representation would be based on the foundation capacity. However, for the jacket-dominated range, this would be based on linear representation of the yield strength. Moreover, for the mixed mode failure region, it would be necessary to represent the response by the surface derived from both the foundation capacity and the yield strength. If the reliability results using the linear response approximations were similar to those predicted by the full response surface, then it could be concluded that the linear approximations were sufficient to represent the specific regions, and there would not be a significant loss of accuracy in their use.

This theory was therefore examined in terms of comparing the reliability results obtained from the full response surface with the reliability results obtained when the linear approximations for the foundations dominated and jacket dominated failures were applied. Figure 80 represents the two linear approximation cases with peak load factor against foundation capacity.

![Figure 80: Peak load factor against foundation capacity showing linear representations of foundations and jacket dominated failures](image)

The reliability index result comparisons are shown in Table 62 for the foundations dominated failure case, and Table 63 for the jacket dominated failure case. The mean of the foundation only failure was set to a value of 6.92 MN, and a range of ±1.26 standard deviations from this mean were considered. It was found that the difference between the reliability indices predicted was ~29%.
The second study examined the reliability index that was obtained when the mean failure mode was well within the jacket dominated failure region, as shown in Figure 80. Results obtained from when the response was based on the yield strength alone, and compared this to when the response was based on both foundation capacity and yield strength over the complete range of foundation capacities. The mean foundation capacity was initially set to a value of 28.4 MN, and a range of ±1.8 standard deviations from this mean were considered. It was found that the difference between the reliability indices predicted was ~22%. If a larger range was studied, with an increased mean of 35 MN, representing a range of ±2.4 standard deviations, it was found that the difference in reliability indices was then ~11%.

Table 63: Reliability results for jacket only failure sensitivity study

These results would indicate that for a sufficiently large range of n within a jacket failure scenario, the closer the reliability result would be to the complete surface results. Further
work is necessary to establish the range of \( n \) required, and to develop this in more detail. At the present time, it is envisaged that a range of ±2.5-3 standard deviations would suffice.

6.7.4 Limits of applicability

Further work is needed in order to validate the hypothesis outlined above. However, at this stage, studies so far would indicate that the linear approximation for the foundation dominated scenario would not be sufficient. As a result the whole range of foundation capacities would be required in order to formulate the response surface, to avoid significant loss of accuracy. The whole range would also be required for the mixed mode failure scenario. For the jacket dominated failure scenario, it would seem that a linear approximation may be appropriate for a range of foundation capacities, say a range of ±2.5 standard deviations would suffice.

This hypothesis could potentially be developed further in order to propose a simplified approach incorporating the coefficient of variation of different parts of the system, depending on the failure mode exhibited or assessed. Despite the understanding that modelling uncertainty is an important parameter to be considered it has not been included in the initial stages of the preliminary simplified approach development. It would clearly need to be addressed if an approach such as that described above were to be developed further. Further investigations would also be necessary to extend the scope of this work to include soils of different types. It should also be noted that this approach has been based on the observations on the study of one jacket structure under one wave approach direction. Further work would be needed to validate this approach for a range of structures and wave approach directions and provide guidelines for the application of the proposed approach.

6.8 Concluding remarks

Investigations into different reliability methodologies used in the offshore industry have been undertaken in detail. Background to the three main approaches has been presented along with details of the key features of the approaches. Two methods have been applied in this study, namely the ‘minimal’ analysis approach and the response surface technique to a steel jacket structure in the southern North Sea. The third method studied was the system analysis approach. WSAAtkins had performed this on the same structure. A confidential report, detailing the analyses undertaken and the results obtained, was made available for this research programme. In the minimal analysis approach, four different wave approach directions were studied. However, the response surface technique was only applied for one wave approach direction – that from platform North. The conclusions drawn therefore
relate only to these conditions and further work would be required in order to affirm the
trends and results noted for additional wave approach directions and other structures. On
this basis, the following conclusions could be made.

The three reliability approaches studied are fundamentally different in the way that the
loading and resistance are represented in the reliability calculation. The minimal analysis
approach is computationally simple and requires only a small number of FE runs to be
performed in order to derive a failure surface compass-diagram. If the distributions are
assumed to be both lognormally represented then standard textbook equations can be used to
derive the reliability index. This method is really only suited to cases characterised by low
resistance uncertainty when compared to the loading uncertainty. If this method is used
within the limits of its applicability, then it has been found to give sufficiently accurate
results, provided that the COVs for the two variables are suitably calibrated.

The response surface method allows either a simple or ‘simplified’ loading distribution
based on elaborate analysis, to interact with a complex surface representation of the
resistance. It can provide an effective method for the derivation of reliability. Whilst
enabling the use of advanced structural analysis in combination with system reliability
methods. Application of this method can also keep the number of pushover runs to a
manageable size, if methods such as the central composite design are applied. RST aims to
capture the key elements of the resistance within a suitable range. However, problems can
be associated with the representation of complex trends and characteristics. Information
about the location of the design point will be an output from the reliability analysis. If this
point falls outside the original surface, then additional runs are required in order to ensure
that the surface is sufficiently accurate in the region of the design point. Changes to the
reliability index and probability of failure will be caused by changes to the distribution of
individual parameters comprising the response surface and care should be taken that the
most accurate representation of these parameters is used. This method enables the use of
advanced structural analysis in combination with system reliability methods.

The system analysis approach is a rigorous approach in which search algorithms are used to
identify the most dominant failure paths. However, it does involve a number of
approximations and it is possible that a dominant failure mode could be overlooked. It is
important to perform such analysis with previous knowledge obtained from a simplified or
deterministic analysis. The method has been successfully applied to the system reliability
analysis of a number of jacket structures, and has been shown to be an efficient search
algorithm. In the comparisons presented here, the ‘system’ method has the most detailed and accurate representation of the environmental loading on the structure. It would, however, be possible to combine more detailed loading models with the minimal and response surface approaches.

A detailed comparison of the three methods was therefore undertaken. Differences in the way in which the loading and resistance are represented within the methods have been described. Differences in the approach to the derivation of the reliability index and probability of failure have also been discussed.

Some preliminary studies have been performed on a method that can be used to determine the level of complexity of the reliability analysis required. This is based on a study of a range of foundation capacities. A minimum of three pushover analyses is required at the mean value, and then at \( \pm n \) standard deviations from the mean. Initial results have indicated that for a sufficiently large range of \( n \) within a jacket dominated failure scenario, a linear approximation (based on the yield strength) can be used instead of a full response surface. More analyses are needed to establish more precisely the range of \( n \) that is required, but it is envisaged that a range of \( > \pm 2.5 \) standard deviations around the mean would be sufficiently accurate.

For the foundations dominated failure scenario, it is proposed that for some structures with certain soil profiles, a linear approximation may be appropriate, provided that the range of \( n \) that is considered is large enough. However, for the structure studied within this research, this range cannot be made sufficiently large in order ensure that accuracy is not lost when a linear approximation (based on foundation capacity) is used, and it is from this standpoint that a full response surface approach is recommended. A full response surface is also necessary when a mixed mode failure is observed, irrespective of the range of foundation capacity studied.
CHAPTER 7.

FINAL FRAMEWORK

7.1 Introduction and background

This chapter presents the revised and updated framework for structural system reliability assessments of offshore platforms. Based on the frameworks presented in Chapter 3, and then incorporating the key findings and experience gained through the studies into key parameters (Chapter 4), key parts of the reliability assessment process (Chapter 5) and the different reliability methods (Chapter 6). For the background on the need for a framework the reader is guided to Chapter 3. Several improvements have been made to the frameworks, and more detailed steps have been provided as a result. These are felt to improve the clarity of the framework and can be used to guide the user towards more consistent reliability assessments. The framework is presented in two distinct formats: a flowchart framework which allows a clear visual representation of the key steps and their interrelation, and a tabular format which allows more detail for each activity to be presented.

Three levels of framework have been developed. A top-level framework shows the outline of the key activities that are required when performing a structural reliability assessment. A second level framework presents a more detailed approach, and provides more information on what each of the individual stages incorporates. A sub-framework level provides the greatest detail for those parts of the process which are of the utmost importance in order to identify and understand where uncertainty is introduced so that more consistent reliability assessments can be produced.

7.2 Presentation of updated framework

7.2.1 Top-level framework

The top-level flowchart, as shown in Chapter 3, was used to show the main elements of the generic framework without going into detail of all the steps required within each stage of the process. This was re-examined in the light of the information obtained from the deterministic and reliability analysis work undertaken. A change was made to divide the
task of ‘capacity and load derivation’ into two separate tasks based on the importance of these activities, as shown in Figure 81. The stage numbers were also added to the flowchart to improve understanding of the subsequent detailed flowcharts and tabular frameworks.

Figure 81: Top-level generic framework flowchart

7.2.2 Second-level framework

As more detailed sections of the framework were developed, it soon became clear that another level of flowchart was needed in order to maintain the clarity of presentation. Figure 82 shows the overall second-level framework.

A new symbol (□) was adopted to indicate where a separate framework needed to be studied for that stage in the system reliability assessment process. One significant change resulting from the detailed studies described in the previous chapters, concerned the requirement of appraising the environmental assessment and the foundation assessment concurrently. It was felt that this was possibly not entirely appropriate and the revised framework shows that these actions should be performed consecutively as in Figure 82.
Figure 82: Second-level generic framework flowchart

- Process
- In/output
- Decision
- Document
- Terminal
- Subframework
The second-level framework (see Figure 82) shows the main steps that are required in order to carry out a structural reliability assessment. The input (shown at the top) is information relating to the details of the structure. The next stage is to make a detailed assessment of the condition of the platform. This stage is described in more detail as a separate framework as shown in Figure 83 and discussed in the subsequent section.

Following this, a decision will be made as to what software package is to be used to undertake the structural assessment, along with the determination of the structural members of the platform. This is then followed by a decision concerning what structural parts are to be included in the model, as shown in Figure 84. Input into this decision will include information on the relevance and importance of parts, for example, a detailed model of the deck may not be necessary and a simplified model for this may be used. An outcome from this stage would include a documentary record of the reasons why certain parts were not included in the analysis model. Once the structural model has been established, an assessment of the environmental loading is required. This is detailed in a separate framework as shown in Figure 85 and is discussed in the subsequent section. Following assessment of the environmental loading, an assessment of the foundations of the structure is undertaken as required. This is detailed in two separate frameworks – one describing the steps required if the foundations are in sand, and the second for clay soil, as shown in Figure 86 and Figure 88 respectively. These figures are based on initial investigations based on a specific platform. It should be noted that further work is needed in order to establish whether the same framework is applicable to other structures.

Accumulation of the platform details, the modelling approach adopted along with the assessment of the environmental loading and foundation capacity results in what has been termed a system analysis model. This contains all the elements necessary to perform a structural system analysis. This model is then used in the reliability analysis stage, which is detailed in two separate frameworks – one for the ‘component’ based approach and the second for the ‘system’ based approach as shown in Figure 89 and Figure 94 respectively. A required output from the reliability analysis stage is to present and interpret the results obtained, and this has been given the symbol of a document to indicate that a summary report should be written. This is then used to produce an overall output of the measure of reliability of the structure, which could be in terms of the probability of failure and the reliability index. It is then necessary to compare the reliability results with any pre-defined targets or acceptance criteria.
The framework shown in Figure 82 can also be presented in a tabular format, as shown in Table 64. This table presents the key stages in the first column and a brief description of the activity in the next column. The flowchart and table should be studied concurrently.

<table>
<thead>
<tr>
<th>Stage</th>
<th>Description</th>
</tr>
</thead>
</table>
| Input | Description of platform structure (from design drawings, defect/damage assessments, condition assessment reports, computer models etc.)
|       | Deadload and live-load parameter values (most applicable to “floater” structures: including buoyancy effects, inertial and dynamic parameters)
|       | Foundation parameter values (soil conditions, pile conditions, group interaction etc.)
|       | Environmental parameter values (from wave height, wave period, current, wind, inertia, drag etc.)
| Stage 1. Assessment of fixing of structure | Assessment of fixing conditions of the structure (in order to determine whether fixed or “floater”)
|       | For “floater” only: Determination of modelling method for “floater” buoyancy effects |
| Stage 2. Modelling of structure | Decision: what software package to use. Determination of structural members (i.e. which members/parts to model, and in what detail) |
| Stage 3. Environmental load assessment | Determination of environmental loads on the structure (from environmental parameters distribution, statistical distribution etc.) |
| Stage 4. Foundation capacity assessment | Determination of foundation capacity & stiffness (from capacity / distribution etc.) according to soil type, assessment method etc. |
| Stage 5. System analysis model derivation | Complete structural analysis using various software options |
| Stage 6. Detailed reliability analysis | Decision as to which reliability methodology to adopt (may be governed by external constraints)
|       | • For “component” based approach: Perform pushover analysis to identify dominant failure modes. Perform either: Minimal analyses, response surface or numerical simulation approaches. Perform number of pushover analyses (determine load-deformation characteristics). Decide on failure criteria e.g. determine ultimate capacity & other failure characteristics, & failure surface if required (from pushover analyses results). Determination of distribution of strength (dependent upon the focus of the study). Integrate distribution of strength with loading on structure. Present assessment of uncertainties.
|       | • For “system” analysis approach: Identify dominant failure modes from search algorithms (derive failure surface if required). Build up structural system, including series of parallel sub-systems if required (dependent upon the focus of the study). Perform reliability analysis. Present assessment of uncertainties. |
| Output | Determination of probability of failure (from study of failure surface in combination with uncertainty descriptions)
|       | Determination of measure of reliability of structure (from probability of failure and uncertainty analysis) |

Table 64: Summary outline framework presented in a tabular format
7.2.3 Presentation of detailed frameworks for each stage

7.2.3.1 Stage 1: assessment of fixing of structure

The detailed individual framework that addresses the assessment of the condition of the platform starts with the input of structural details, as shown in Figure 83.

![Figure 83: Detailed generic framework flowchart - assessment of platform conditions](image)

Figure 83: Detailed generic framework flowchart - assessment of platform conditions

- Process
- In/output
- Decision
- Document
- Terminal
- Subframework

The next stage is then to follow the appropriate route for either the design of a new platform or the reassessment of an existing platform. If the reliability is to be derived for a new platform at the design stage, then platform design details are required, which could be in the form of detailed drawings and fabrication detail reports. If reassessment is to be carried out
then inspection reports, weld defect assessments and information relating to specific damage incidents, such as ship collision will be required. This information will then be assessed in order to ascertain the condition of the structure, for example, where the actual structure now differs from the original design.

Once this has been undertaken then an assessment is made of the fixing of the structure to confirm whether it has fixed foundations or whether it is a ‘floater’ structure such as a semi-submersible. At this stage, deadload parameters, which will include buoyancy effects, will need to be studied, along with examination of inertial and dynamic loading. For a ‘floater’ structure the buoyancy effects and their distributions will need to be determined, which will be influenced by assumptions, judgement and the knowledge of the user. The uncertainty associated with these parameters will need to be derived and these should be documented. A decision then follows on the method by which these buoyancy effects should be modelled, and again, information on the associated uncertainty should be produced. This overall full description of the structure will then be taken forward to model the structure. This has been described earlier in section 7.2.2.

The tabular framework developed that relates to the platform assessment stage is shown in Table 65. It is intended that the flowchart and table should be studied together.

<table>
<thead>
<tr>
<th>Stage 1. Assessment of fixing of structure</th>
<th>Brief description of activity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Input 1</td>
<td>Platform structural details</td>
</tr>
<tr>
<td>Step 1.1a (for new/design case)</td>
<td>For the new/design case: obtain all detailed design drawings etc</td>
</tr>
<tr>
<td>Step 1.1b (for old/reassessment case)</td>
<td>For old/reassessment case: obtain design drawings if possible, along with inspection reports, weld defect assessments, specific damage/defect reports where available, then make full condition assessment of structure.</td>
</tr>
<tr>
<td>Step 1.2</td>
<td>Assessment of structure fixing conditions i.e. fixed or floating. Includes input of deadload parameters including buoyancy effects and derivation of inertial and dynamic loading parameters.</td>
</tr>
<tr>
<td>Step 1.3</td>
<td>For &quot;floater&quot; structures, determine buoyancy effects etc. and the distribution, using the assumptions, judgement and knowledge of user. Determine estimates of the associated uncertainty.</td>
</tr>
<tr>
<td>Step 1.4</td>
<td>Decide on modelling method for ‘floater’ buoyancy effects and determine associated uncertainty</td>
</tr>
<tr>
<td>Output 1</td>
<td>Full structural detailed model of the design or current conditions of the platform having taken into account changes since installation.</td>
</tr>
</tbody>
</table>

Table 65: Detailed breakdown table Stage 1: Assessment of platform conditions
7.2.3.2 Stage 2: modelling of the structure

The second-level generic framework flowchart as shown in Figure 82 includes the details for the Stage 2: modelling of the structure. This involves a decision as to what software package to use, which may be governed by external constraints. The structural members that need to be determined will be identified. This is then followed by a decision concerning what structural parts are to be included in the model. For example, it may not be necessary to model the deck and all its equipment in detail. This will be decided based on the relevance or importance of the parts. Valid reasons why certain parts are not included should be produced. A decision then follows as to how the parts are to be modelled - the user will influence this. Justification for the modelling method chosen should also be produced.

Table 66 shows the tabular framework for the second stage. The first column shows input, output and the individual steps required, while the second column presents a brief description of the activity to be undertaken.

![Figure 84: Extract from second-level generic framework flowchart
Stage 2 modelling of structure](image)
### Stage 2: Modelling of Structure

<table>
<thead>
<tr>
<th>Brief description of activity</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Input 2</strong> From Stage 1: description of platform structure &amp; fixing conditions</td>
</tr>
<tr>
<td>(from design drawings, computer models etc.)</td>
</tr>
<tr>
<td><strong>Step 2.1</strong> Decision as to what software package to use</td>
</tr>
<tr>
<td>(may be governed by external constraints)</td>
</tr>
<tr>
<td><strong>Step 2.2</strong> Determination of structural members</td>
</tr>
<tr>
<td><strong>Step 2.3</strong> Assessment of relevance or importance of structural parts</td>
</tr>
<tr>
<td>(e.g. is a detailed deck necessary?)</td>
</tr>
<tr>
<td><strong>Step 2.4</strong> Decision as to what structural parts are to be included in the model</td>
</tr>
<tr>
<td><strong>Step 2.5</strong> Presentation of valid reasons why certain parts are not included</td>
</tr>
<tr>
<td><strong>Step 2.7</strong> Decision as to how parts are to be modelled</td>
</tr>
<tr>
<td><strong>Step 2.8</strong> Presentation of the justification for the modelling method chosen</td>
</tr>
<tr>
<td><strong>Output 2</strong> Full model of structure appropriate to &amp; specific to the current assessment</td>
</tr>
<tr>
<td>being undertaken</td>
</tr>
</tbody>
</table>

**Table 66:** Detailed breakdown table Stage 2: Modelling of structure

#### 7.2.3.3 Stage 3: Environmental Load Assessment

The detailed framework relating to the assessment of the environmental loading is shown in Figure 85 and Table 67. This stage follows the steps required to fully describe the platform and those decisions concerning the modelling and inclusion of structural parts. It was noted in the review described in Chapter 2 that some elements that constitute the environmental loading are much more significant and tend to dominate. It was found that the wave height was the most important parameter, followed by the wave period. The drag coefficient used within Morison's equation is also an important factor and it was noted that its uncertainty should not be ignored. The storm current, which can contribute 10% of the wave induced forces, is less important. However, for the inertia coefficient it was found that a deterministic value could be used, and as far as marine growth was concerned, this too could be modelled as deterministic.

In order to improve this section of the framework, the individual inputs into the environmental load assessment process have been itemised, and these are presented in sequence of their perceived importance and significance. The framework presented in Chapter 3 grouped all environmental parameters under a combined input and it was felt that the changes described above would improve the clarity of the framework.
Stage 3. Environmental load assessment

<table>
<thead>
<tr>
<th>Brief description of activity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Input 3</td>
</tr>
<tr>
<td>Step 3.1</td>
</tr>
<tr>
<td>Step 3.2</td>
</tr>
<tr>
<td>Step 3.3 a, b, c</td>
</tr>
<tr>
<td>Step 3.4</td>
</tr>
<tr>
<td>Step 3.5</td>
</tr>
<tr>
<td>Step 3.6</td>
</tr>
<tr>
<td>Step 3.7</td>
</tr>
<tr>
<td>Output 3</td>
</tr>
</tbody>
</table>

Table 67: Detailed breakdown table Stage 3: Environmental load assessment
7.2.3.4 Stage 4: foundation capacity assessment

There are two detailed frameworks that describe the steps required in order to make a full assessment of the foundation conditions of the structure. Figure 86 and Figure 87 show the first framework that presents the steps needed for assessment of foundations in sand, and Figure 88 shows the steps that must be undertaken for foundations in clay. Table 68 and Table 69 show the tabular frameworks for the assessment of foundations in sand and clay respectively.

Foundations in sand

For the case of foundations in sand, it is first necessary to perform a full pushover analysis of the structure and to make an assessment of the dominant failure mode, based on using the API recommendations for stiffness and capacity. The API recommendations, which are used as a starting point for foundation capacity assessment, are those which have been widely used in the design of the majority of North Sea structures currently installed. It will then be necessary to perform two additional pushover analyses at values of ±n standard deviations from the mean of the foundation capacity, in order to aid an assessment of the failure mode over a range of foundation capacities.

These additional analyses at ±n standard deviations are recommended as a screening test to assess the level of complexity of the resistance model required in the reliability analyses. The value of n will be different depending on the initial failure mode obtained at mean values (i.e. jacket or foundations failure). Further work is needed to derive definitive values for n but initial investigations undertaken within this research indicate values in the range of ±2.5-3 may be appropriate.

A plot of peak load factor (or ultimate capacity) of the structure should then be plotted against foundation capacity. A trendline should then be fitted to the three data points. Depending on the shape and gradient of this trendline, an assessment can be made of the failure mode for the range of foundation capacities. There will be three possible outcomes from this.

The first is where the trendline fitted is linear and exhibits a near horizontal gradient. In this case, a jacket dominated failure scenario can be deduced for the range studied. In this scenario, the foundations do not fail and where failure is located in the jacket, it is suggested that the foundations can be assumed to be deterministic. This means that in the subsequent reliability assessment process, the resistance model can be derived by focusing on the yield strength of the material, along with modelling uncertainty. The second
scenario is where the trendline fitted is linear and exhibits a positive gradient. From this it can be deduced that over the range a foundations dominated failure scenario is exhibited. The third scenario is where it is not possible to fit a linear trendline through the three data points. From this it can be deduced that over the range, a mixed mode failure is exhibited.

In the last two scenarios, where failure is in both the jacket and the foundations, and where the failure is foundation dominated, in the ensuing reliability analysis work the foundations uncertainty cannot be ignored.

A decision as to how the foundations are to be assessed then follows. Two key methods are considered: the API recommendations and the newer IC method. If the API method is used, it has been found that the result can be over-conservative and not sufficiently accurate. This method is not the recommended method for use in reliability assessments. If the IC method is used, there are three possible scenarios when a suitable method is used to determine the dominant failure mode.

Figure 86: Sub-framework flowchart – foundation capacity assessment for sands part 1
The first would be that the mixed mode failure might shift to become a jacket-dominated failure as the assessment process is changed. If this is the case, then again it is suggested that foundations can be assumed to be deterministic in the subsequent reliability analysis.

The second would be a case where the API assessed foundation-dominated case is now shifted to a mixed mode case when the IC method is applied. In this case, the resistance model in the subsequent reliability assessment can be based on a calibration in terms of both the yield strength and the foundation capacity.

The final scenario is where a foundation failure predicted by the API method would remain as a foundation failure when assessed by the IC method. In this case, the resistance model in the subsequent reliability assessment will be dominated by the foundation capacity only, and the effects of the yield strength will be negligible and can be ignored, whilst also taking into account modelling uncertainty.

For those cases where the foundation capacity has to be taken into account (for both the mixed mode failures and the foundation-dominated failures) a decision follows as to which additional foundation factors need to be considered. The flowchart then branches to represent both the cyclic loading effect and the ageing effect. Under each effect there is a choice of whether the factor needs to be applied. The cyclic loading would provide a degradation and the ageing would provide a benefit in terms of the pile axial capacity predicted.

For the case where there would be no cyclic loading effect in combination with long term ageing, this is considered to be the 'best case' combination of the two effects. This may mean that the failure mode may shift from a mixed mode to a jacket dominated failure scenario. In this instance, the following reliability assessment could then be calibrated by using the yield strength and the modelling uncertainty.

For the case where there would be a cyclic loading effect in combination with short term i.e. no ageing, this is considered to be the 'worst case' combination of the two effects. In this case detailed foundation reliability assessment would need to be undertaken, for example by using a response surface approach, which would be calibrated in terms of the yield strength and the foundation capacity, along with modelling uncertainty.
Figure 87: Sub-framework flowchart – foundation capacity assessment for sands part 2
<table>
<thead>
<tr>
<th>Stage 4b. Foundation capacity assessment for sands</th>
<th>Brief description of activity</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Input 4</strong></td>
<td>Obtain details of the design of the platform foundations and as much information on the soil conditions at the site as available. Soil data should include CPT test data, density, grain size etc. if possible.</td>
</tr>
<tr>
<td><strong>Step 4b.1</strong></td>
<td>For sand soils: perform pushover analysis to determine dominant failure mode (using API stiffness and mean capacity values).</td>
</tr>
<tr>
<td><strong>Step 4b.2</strong></td>
<td>Perform two additional analyses at ±n standard deviations from the mean foundation capacity and derive dominant failure mode of each.</td>
</tr>
<tr>
<td><strong>Step 4b.3</strong></td>
<td>Create a graph plot of peak load factor against foundation capacity and fit trendline to the three data points.</td>
</tr>
<tr>
<td><strong>Step 4b.4a</strong></td>
<td>If the trendline fitted is linear, and has a near horizontal gradient, it can be deduced that over the range of foundation capacity, that the failure is jacket dominated.</td>
</tr>
<tr>
<td><strong>Step 4b.5</strong></td>
<td>For a jacket dominated failure scenario: the subsequent reliability analysis can be calibrated on the yield strength of the material of the structure (and modelling uncertainty), further detailed assessment of foundations is not required.</td>
</tr>
<tr>
<td><strong>Step 4b.4b</strong></td>
<td>If the trendline fitted is linear, and has a positive gradient, it can be deduced that over the range of foundation capacity, that the failure is foundations dominated.</td>
</tr>
<tr>
<td><strong>Step 4b.4c</strong></td>
<td>If a linear trendline cannot be fitted to the data, it can be deduced that over the range of foundation capacity, that the failure is of mixed mode.</td>
</tr>
<tr>
<td><strong>Step 4b.5</strong></td>
<td>For a mixed mode failure or for a foundations dominated failure, then foundations cannot be ignored, and further detailed assessments are required.</td>
</tr>
<tr>
<td><strong>Step 4b.6</strong></td>
<td>Decide on what method is to be adopted for the assessment of the foundations?</td>
</tr>
<tr>
<td><strong>Step 4b.7</strong></td>
<td>If the API method is selected: this has been found to be very conservative and not sufficiently accurate and thus not recommended.</td>
</tr>
<tr>
<td><strong>Step 4b.8a</strong></td>
<td>If the IC method is selected: using a suitable technique (such as that which uses three pushover analyses at ±2 standard deviations and the mean) derive the dominant failure mode.</td>
</tr>
<tr>
<td><strong>Step 4b.8b</strong></td>
<td>For the jacket-dominated failure and the mixed mode failure observed (when using the API assessment in Step 4.1) are likely to exhibit jacket only failure when assessed with the IC method. The subsequent reliability analysis can be calibrated on the yield strength of the material of the structure with modelling uncertainty, and further detailed assessment of the foundations is not required.</td>
</tr>
<tr>
<td><strong>Step 4b.8c</strong></td>
<td>For the IC method: the foundation-dominated failure observed (when using the API assessment in Step 4.1) is likely to exhibit a mixed mode failure when assessed with the IC method. The subsequent reliability analysis can be calibrated on both the yield strength and the foundation capacity combined, with the modelling uncertainty, and further detailed assessment of the foundations is not required.</td>
</tr>
<tr>
<td><strong>Step 4b.8d</strong></td>
<td>For the IC method: the foundation-dominated failure observed (when using the API assessment in Step 4.1) may exhibit foundation only failure when assessed with the IC method. The subsequent reliability analysis can be calibrated on the foundation capacity with the modelling uncertainty.</td>
</tr>
</tbody>
</table>
For the cases that had a mixed mode or a foundation-dominated failure, decide what other effects need to be examined, if any.

If cyclic loading is to be considered: two cases are to be assessed: with and without the degradation of cyclic loading.

If ageing is to be considered: two cases can be assessed: with and without the beneficial effects of ageing.

For the ‘worst’ case assessed where there is cyclic loading and no ageing effect, then a mixed mode failure initially exhibited when the IC method was used may now remain, or may now become a foundation dominated scenario. The subsequent reliability analysis can be calibrated on yield strength and foundation capacity, in conjunction with the modelling uncertainty.

For the ‘best’ case assessed where there is no cyclic loading and there is ageing, then the mixed failure mode may shift to jacket dominated scenario. The subsequent reliability analysis can be calibrated on yield strength and modelling uncertainty.

Determine uncertainty associated with the foundation parameters.

Full model of foundations for sand soils and associated uncertainty.

Table 68: Detailed breakdown table Stage 4b: foundation capacity assessment for sands

Foundations in clay

For the case of an assessment of piled foundations in clay, as shown in Figure 88, it is first necessary to perform a full pushover analysis of the structure. This is in order to make an assessment of the dominant failure mode using a suitable technique, based on using the API recommendations for stiffness and capacity. There will be three outcomes from this – jacket-dominated, mixed mode or foundation-dominated failure modes.

For each of the three cases the subsequent reliability assessment must include a detailed assessment of the foundations. This is necessary for each of the three modes, as for certain clays the API assessment may be unconservative.

A decision then follows as to what method of assessment is to be adopted either API or IC. If the API method is considered, it has been found that the result can be unconservative and not sufficiently accurate and thus this method is not the recommended method for use in reliability assessments.

If the IC method is used, there are four scenarios. Firstly, a jacket-dominated failure predicted over a range of foundation capacities by API may remain a jacket-dominated failure when the IC method is used. Here the subsequent reliability analysis would be calibrated on the yield strength and modelling uncertainty. Secondly, a jacket-dominated failure predicted by API may become a mixed mode failure when the IC method is used. Here the resistance model in the reliability analysis would be calibrated in terms of both the yield strength and the foundation capacity, along with the modelling uncertainty. Thirdly, a
mixed mode failure predicted by API may become a foundation-dominated failure by IC. Finally a foundation-dominated failure may remain as such whether the API or IC method is used. For these last two scenarios, the reliability analysis would need to be calibrated on foundation capacity and modelling uncertainty.

Figure 88: Sub-framework flowchart – foundation capacity assessment for clays

- Process
- In/output
- Decision
- Document
- Terminal
- = subframework
**Stage 4a. Foundation capacity assessment for clays**

<table>
<thead>
<tr>
<th>Brief description of activity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Input 4 Obtain details of the design of the platform foundations and as much information on the soil conditions at the site as available. Soil data should include CPT test data, density, grain size etc. if possible.</td>
</tr>
<tr>
<td>Step 4a.1 For clay soils: perform pushover analysis to determine dominant failure mode (using API stiffness and mean capacity values).</td>
</tr>
<tr>
<td>Step 4a.2 Perform additional analyses for a range of foundation capacity and derive dominant failure mode of each.</td>
</tr>
<tr>
<td>Step 4a.3 For clay soils: decide on what is the method of assessment for the foundations.</td>
</tr>
<tr>
<td>Step 4a.4 a If the API RP2A recommendations are to be used: API could be unconservative, and not sufficiently accurate, thus not recommended</td>
</tr>
<tr>
<td>Step 4a.4 b If the IC method equations are to be used: perform pushover analysis to derive dominant failure mode.</td>
</tr>
<tr>
<td>Step 4a.5 a Using a suitable technique (such as that which uses three pushover analyses at ±2 standard deviations and the mean) derive the dominant failure mode.</td>
</tr>
<tr>
<td>Step 4a.5 b If jacket-only failure is exhibited then can calibrate the subsequent reliability assessment based on yield strength alone.</td>
</tr>
<tr>
<td>Step 4a.5 c If mixed mode failure consisting of both jacket and foundation failure is exhibited then can calibrate the subsequent reliability assessment based on both the yield strength and the foundation capacity.</td>
</tr>
<tr>
<td>Step 4a.5 d If foundation-only failure is exhibited then can calibrate the subsequent reliability assessment based on foundation capacity alone.</td>
</tr>
<tr>
<td>Step 4a.6 Determine uncertainty associated with the foundation parameters assessed.</td>
</tr>
<tr>
<td>Output 4a Full model of foundation capacity for clay soils and its associated uncertainty.</td>
</tr>
</tbody>
</table>

Table 69: Detailed breakdown table Stage 4a: Foundation capacity assessment for clay soils

### 7.2.3.5 Stage 5: system analysis model derivation

Table 70 describes the key inputs that go into the formation of the system analysis model, once the platform assessment has been undertaken, the environmental loading assessed and the foundations studied in detail. The output of this collation stage is a detailed model, which can be used to represent the platform in terms of structure, loading and resistance.

<table>
<thead>
<tr>
<th>Brief description of activity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Input 5 Obtain detailed presentation of structural design, environmental loading and foundation capacity assessment of the platform.</td>
</tr>
<tr>
<td>Step 5.1 Collate all information including individual parametric details. Collate all information on platform, loading and resistance.</td>
</tr>
<tr>
<td>Output 5 A detailed model representing the platform, environment and structural resistance.</td>
</tr>
</tbody>
</table>

Table 70: Detailed breakdown table Stage 5: System analysis model derivation
7.2.3.6 Stage 6: detailed reliability analysis

The detailed generic flowchart for the reliability assessment process for a ‘component’ based approach is shown in Figure 89 and Table 71. The term ‘component’ here implies that the analysis is based on the assumption that the structure is to be considered as one component rather than the ‘system’ approach where the assumption is that the structure is comprised of a number of components which make up the overall system. A more detailed separate framework has been developed for each of the ‘component’ methods. Three frameworks have been prepared for the minimal analysis (see Figure 90 and Table 72), response surface (see Figure 91 and Figure 92 with Table 73) and the numerical simulation approaches (see Figure 93 and Table 74). The process for a ‘system’ based approach is shown in Figure 94.

In the ‘component’ based approach in Figure 89, the first task is a pushover analysis on the structure to ascertain the dominant failure modes. Following this, one of the three key methods will be applied – the ‘minimal’ analysis, or response surface or numerical simulation approaches in order to derive a measure of reliability in terms of the probability of failure and reliability index. A comparison against pre-defined targets and acceptance criteria is then performed and it is necessary to present, understand and interpret the results.

![Figure 89: Detailed generic framework flowchart – reliability ‘component’ assessment](image-url)
Stage 6a.
‘Component’ based reliability analysis

<table>
<thead>
<tr>
<th>Brief description of activity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Input 6</td>
</tr>
<tr>
<td>Step 6.1</td>
</tr>
<tr>
<td>Step 6a.2</td>
</tr>
<tr>
<td>Step 6a.3</td>
</tr>
<tr>
<td>Step 6a.4</td>
</tr>
<tr>
<td>Step 6a.5</td>
</tr>
<tr>
<td>Step 6a.6</td>
</tr>
<tr>
<td>Output 6a</td>
</tr>
</tbody>
</table>

Table 71: Detailed breakdown table Stage 6a:
Reliability analysis: ‘component’ based approach

Minimal analysis approach

Table 70 shows the ‘component’ based approach that can be performed using three different methods. The first method is the ‘minimal’ analysis as shown in Figure 90. The first stage will be to perform a number of non-linear pushover analyses, for example for eight different wave approach directions. A calculation of the peak load factor for each direction must then be performed, in order that the ultimate capacity of the structure can be derived. From this it will be possible to identify the two worst wave directions, for those waves which cause the lowest ultimate capacity to be realised. A decision on the failure criteria then follows. Then a decision must be made on what distributions are to be used to represent the loading on the structure and its resistance. This will be based on previous analyses, assumptions and judgement of the user.

The loading will be represented by a simple distribution, where the statistics of this variable can be determined from that of the wave height alone, on the assumption that a simple relationship exists between the wave height and the member force. Alternatively, the loading model may be derived through a rigorous methodology processing environmental load statistical data, therefore, providing a more accurate representation of the load distribution.
The resistance will also be represented by a simple distribution, with the mean being derived from the ultimate capacity of the structure. Associated COV will be derived from previous studies and available literature.

The uncertainty associated with both the loading and resistance will need to be determined. The next task is then to integrate the distribution of the strength (resistance) with the distribution of the loading in order to obtain a probability of failure for the structure. Presentation of the uncertainties will be needed. Assuming independence of the system failure event for different wave approach directions, the overall probability of failure will be derived.

**Figure 90: Sub-framework flowchart – reliability assessment by minimal analysis technique**

- Process
- In/output
- Decision
- Document
- Terminal
- Subframework

Vanessa J Forbes
PhD Thesis
Page 248
Response surface technique

Figure 89 shows the ‘component’ based approach that can be performed using three different methods. The second such method is the response surface technique as shown in Figure 91 and Figure 92. The framework for this method has been divided into two sub-frameworks for clarity of presentation.

Figure 91: Sub-framework flowchart: reliability assessment by response surface technique 1

The first task to be carried out is to obtain or perform a number of preliminary non-linear pushover analysis results. A succession of three decisions then follows where the assumptions, judgement and knowledge of the user will all affect the outcome. These decisions address what factors need to be assessed according to the focus of the study, what RST method is to be adopted (e.g. central composite design) and precisely what range the
individual parameters need to be represented within the response surface. A set of observations will need to be performed and the precise details of these need to be drawn up. Once the ‘experiment’ has been derived from a number of observations, a series of pushover analyses are then carried out for each observation point. The peak load factor results for the observations are then collated and plotted in order that a visual examination can be made. A decision is then made as to the general form of the equation that will represent the surface. The following decision then addresses how the modelling uncertainty is to be incorporated into the surface. Again, the user will have influence on these decisions.

Figure 92: Sub-framework flowchart: reliability assessment by response surface technique – part 2
Calibration of the surface is then performed by deriving the individual constants that form the equation. This could be by an iterative solution based on minimising the sum of the error squared. An assessment of the bias of the equation is then performed, in order to verify that the form of the equation adequately represents the characteristics of the surface and that no unaccounted trends remain. If there is no trend in the bias, then the equation is acceptable. However, if the bias exhibits any kind of trend the equation representing the surface will need to be redefined. Distributions are then selected which will represent the parameters within the surface equation, before the reliability analysis is performed to obtain a value for the probability of failure and reliability index. Information is also obtained from the reliability analysis that describes the location of the design point. In order to verify the accuracy of the surface an additional pushover analysis is performed at this design point location. An assessment is then made as to whether the predicted response from the surface and the measured results from the pushover analysis are within an acceptable tolerance. This could be of the order of 5%. If the difference is less than the tolerance, the reliability results will be considered acceptable, but if the difference exceeds the tolerance then additional runs must be performed and the surface refitted. The procedure should be repeated until eventually acceptable results are achieved.

**Numerical simulation technique**

Figure 89 shows the ‘component’ based approach that can be performed using three different methods. The third method is based on a numerical simulation approach as shown in Figure 93. The first task to be carried out is to perform a number of preliminary non-linear pushover analyses. A decision then follows where the assumptions, judgement and knowledge of the user will affect the outcome. The question to be answered is what factors will need to be addressed according to the focus of the study. A number of numerical simulation analyses are then required before a decision is made on what failure criteria to adopt. The ultimate capacities (and other failure characteristics) are then determined and a failure surface constructed if required. The distribution of the loading and the strength are then determined and integrated to obtain the probability of failure of the structure. An assessment of the uncertainties will also be required.

This framework for the numerical simulation approach is not as detailed as those for the minimal analysis and response surface techniques, as this method was not specifically studied within this research programme (see Chapter 3 for more details on the numerical simulation approach applied by [Sigurdsson et al., 1994]).
Step 6.3 Decide on which technique is to be used to derive the reliability.

Step 6a.3 Part 1.1 For the 'minimal' analysis technique: Perform a number of non-linear pushover analyses, for example, for eight different wave approach directions.

Step 6a.3 Part 1.2 Collate peak load factor results, and identify the two worst wave approach directions.

Step 6a.3 Part 1.3 Using the assumptions, judgement and knowledge of the user, decide on failure criteria to be adopted.

Step 6a.3 Part 1.4 Using the assumptions, judgement and knowledge of the user, decide on the functions that are to be adopted to represent the loading and the resistance of the structure.

Step 6a.3 Part 1.5 In the case of the loading, this will be based on a simple distribution, with the statistics based on the wave height alone, due to its overall dominance on the environmental loading.

Step 6a.3 Part 1.6 Determine uncertainty associated with the environmental loading distribution to be applied.
<table>
<thead>
<tr>
<th>Step 6a.3 Part 1.7</th>
<th>In the case of the resistance, this will also be based on a simple distribution. An appropriate COV will need to be selected.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Step 6a.3 Part 1.8</td>
<td>Determine uncertainty associated with the resistance distribution to be applied.</td>
</tr>
<tr>
<td>Step 6a.3 Part 1.9</td>
<td>Integrate the distribution of the resistance (strength) with the loading applied to the structure to obtain a measure of reliability in terms of reliability index and probability of failure.</td>
</tr>
<tr>
<td>Step 6a.3 Part 1.10</td>
<td>Present an assessment of the incorporated uncertainties including modelling uncertainty if necessary.</td>
</tr>
<tr>
<td>Step 6a.3 Part 1.11</td>
<td>Assuming independence of system failure event for each wave approach direction, derive an overall probability of failure for the structure at its location.</td>
</tr>
<tr>
<td>Step 6a.4</td>
<td>Obtain a measure of the reliability of the structure. Present understand and interpret the results.</td>
</tr>
<tr>
<td>Step 6a.5</td>
<td>Perform comparison of reliability with pre-defined targets and acceptance criteria.</td>
</tr>
<tr>
<td>Step 6a.6</td>
<td>Determine uncertainty associated with the reliability derived.</td>
</tr>
<tr>
<td>Output 6a</td>
<td>A measure of the reliability of the structure (e.g. probability of failure and reliability index), which has been compared to targets and acceptance criteria, with assessment of associated uncertainty.</td>
</tr>
</tbody>
</table>

Table 72: Sub-table Stage 6a Part 1: Reliability analysis using ‘component’ based approach using the ‘minimal’ analysis technique

<table>
<thead>
<tr>
<th>Stage 6a Part 2. Response surface technique</th>
<th>Brief description of activity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Step 6.3</td>
<td>Decide on which technique is to be used to derive the reliability.</td>
</tr>
<tr>
<td>Step 6a.3 Part 2.1</td>
<td>For the response surface technique: obtain/perform a number of nonlinear pushover analyses.</td>
</tr>
<tr>
<td>Step 6a.3 Part 2.2</td>
<td>Using assumptions, judgement and knowledge of the user decide on what factors need to be assessed relevant to the focus of the study.</td>
</tr>
<tr>
<td>Step 6a.3 Part 2.3</td>
<td>Using assumptions, judgement and knowledge of the user decide on what RST method is to be adopted for the study e.g. CCD.</td>
</tr>
<tr>
<td>Step 6a.3 Part 2.4</td>
<td>Using assumptions, judgement and knowledge of the user decide on what the ranges are that need to be assessed for the individual parameters chosen to represent the response surface.</td>
</tr>
<tr>
<td>Step 6a.3 Part 2.5</td>
<td>Present an assessment of the incorporated uncertainties including modelling uncertainty if necessary.</td>
</tr>
<tr>
<td>Step 6a.3 Part 2.6</td>
<td>Draw up the set of observations that will need to be performed to produce the response surface.</td>
</tr>
<tr>
<td>Step 6a.3 Part 2.7</td>
<td>Perform the observations required by carrying out a pushover analysis for each observation.</td>
</tr>
<tr>
<td>Step 6a.3 Part 2.8</td>
<td>Collate and examine the peak load factor results, and plot them to allow visual interpretation of the data.</td>
</tr>
<tr>
<td>Step 6a.3 Part 2.9</td>
<td>Using assumptions, judgement and knowledge of the user decide on the general form of the equation, which will represent the surface.</td>
</tr>
<tr>
<td>Step 6a.3 Part 2.10</td>
<td>Calibrate the surface by deriving constants for the equation, for example, by the use of an iterative least squares approach.</td>
</tr>
<tr>
<td>Step 6a.3 Part 2.11</td>
<td>Decide whether the resulting bias exhibits any unaccounted for trends.</td>
</tr>
<tr>
<td>Step 6a.3 Part 2.12</td>
<td>If the bias DOES show any trends, the form of the surface equation must be redefined. Go back Step 5a.3 Part 2.2 and repeat process.</td>
</tr>
</tbody>
</table>
Step 6a.3 Part 2.13 | If the bias does NOT show any trends, the surface equation can be input into the reliability analysis to represent strength of the structure.

Step 6a.3 Part 2.14 | Using assumptions, judgement and knowledge of the user decide on the distributions that will be adopted to represent the individual parameters used to form the response surface.

Step 6a.3 Part 2.15 | Present an assessment of the incorporated uncertainties including modelling uncertainty if necessary.

Step 6a.3 Part 2.16 | Integrate the distribution of environmental loading applied to the structure with the response surface to obtain a measure of reliability in terms of reliability index and probability of failure.

Step 6a.3 Part 2.17 | Obtain the location of the 'design point' from the reliability analysis.

Step 6a.3 Part 2.18 | Verify the surface by performing one additional pushover analysis at the design point location, and compare results obtained to those predicted by the surface.

Step 6a.3 Part 2.19 | Assign a value for acceptable tolerance between the pushover result and the surface prediction at the design point location.

Step 6a.3 Part 2.20 | Using assumptions, judgement and knowledge of the user assess whether the surface prediction is deemed to be adequate – is the difference between the observed result and the surface prediction at the design point within a predefined tolerance?

Step 6a.3 Part 2.21 | If the result is outside the tolerance, then go to Step 5a.3 Part 2.10 and repeat the process as necessary.

Step 6a.3 Part 2.22 | If the result is within the tolerance, then the reliability results can be seen to be acceptable.

Step 6a.4 | Obtain a measure of the reliability of the structure. Present understand and interpret the results.

Step 6a.6 | Perform comparison of reliability with pre-defined targets and acceptance criteria.

Step 6a.6 | Determine uncertainty associated with the reliability measure derived.

Output 6a | A measure of the reliability of the structure (e.g. probability of failure and reliability index), which has been compared to targets and acceptance criteria, with assessment of associated uncertainty.

Table 73: Sub-table Stage 6a Part 3: Reliability analysis using 'component' based approach using the response surface technique

<table>
<thead>
<tr>
<th>Stage 6a Part 3. Numerical simulation technique</th>
<th>Brief description of activity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Step 6.3</td>
<td>Decide on which technique is to be used to derive the reliability.</td>
</tr>
<tr>
<td>Step 6a.3 Part 3.1</td>
<td>For the numerical simulation technique: Perform a number of non-linear pushover analyses.</td>
</tr>
<tr>
<td>Step 6a.3 Part 3.2</td>
<td>Using assumptions, judgement and knowledge of the user decide on what factors (stochastic variables) need to be assessed relevant to the focus of the study.</td>
</tr>
<tr>
<td>Step 6a.3 Part 3.3</td>
<td>Perform a number of numerical simulations (realisations).</td>
</tr>
<tr>
<td>Step 6a.3 Part 3.4</td>
<td>Using assumptions, judgement and knowledge of the user decide on failure criteria to be adopted.</td>
</tr>
</tbody>
</table>
**Structural System Reliability Framework For Fixed Offshore Platforms**

<table>
<thead>
<tr>
<th>Step 6a.3 Part 3.5</th>
<th>Determine ultimate capacity and other failure characteristics and determine failure surface if required.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Step 6a.3 Part 3.6</td>
<td>Determine distribution of resistance (strength) of members and/or structure.</td>
</tr>
<tr>
<td>Step 6a.3 Part 3.7</td>
<td>Integrate the distribution of the resistance (strength) with the loading applied to the structure to obtain a measure of reliability in terms of reliability index and probability of failure.</td>
</tr>
<tr>
<td>Step 6a.3 Part 3.8</td>
<td>Present an assessment of the incorporated uncertainties including modelling uncertainty if necessary.</td>
</tr>
<tr>
<td>Step 6a.4</td>
<td>Obtain a measure of the reliability of the structure. Present understand and interpret the results.</td>
</tr>
<tr>
<td>Step 6a.5</td>
<td>Perform comparison of reliability with pre-defined targets and acceptance criteria.</td>
</tr>
<tr>
<td>Step 6a.6</td>
<td>Determine uncertainty associated with the reliability derived.</td>
</tr>
<tr>
<td>Output 6</td>
<td>A measure of the reliability of the structure (e.g. probability of failure and reliability index), which has been compared to targets and acceptance criteria, with assessment of associated uncertainty.</td>
</tr>
</tbody>
</table>

Table 74: Sub-table Stage 6a Part 3: Reliability analysis using 'component' based approach using the numerical simulation technique

**System based approach**

In the ‘system’ based approach as shown in Figure 94, the first task that will be performed is to identify dominant failure modes. This may be performed by the use of search algorithms. At this stage it is necessary to decide on the failure criteria and it will also be here that the user will influence the analysis. The assumptions and judgement made, along with the knowledge of the user, will contribute to the actions performed.

The following task is to build up a structural system to represent the whole structure, which will comprise of series and parallel subsystems. A decision at this stage will involve what factors need to be assessed in terms of being relevant to the focus of the study. Again the user will have some influence at this stage. After this, a component reliability analysis will be undertaken, which leads to a calculation of the system and sensitivity measures. There will be some degree of uncertainty at this stage and it is necessary for the user to determine this.

Once this method has been completed, the measure of reliability in terms of the probability of failure and reliability index will be derived. Once derived, a comparison against pre-defined targets and acceptance criteria is needed. At this final stage, it will be necessary to present, understand and interpret the results in a summary report.
Structural System Reliability Framework For Fixed Offshore Platforms

Figure 94: Detailed generic framework flowchart – reliability ‘system’ assessment

<table>
<thead>
<tr>
<th>Stage 6b. ‘System’ based reliability analysis</th>
<th>Brief description of activity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Input 6</td>
<td>Obtain system analysis model based on detailed presentation of structural design, environmental loading and foundation capacity assessment of the platform.</td>
</tr>
<tr>
<td>Step 6.1</td>
<td>Decide on which reliability approach is to be adopted, in terms of a ‘component’ based or ‘system’ based approach.</td>
</tr>
<tr>
<td>Step 6b.2</td>
<td>For the ‘system’ based approach, perform pushover analysis to identify dominant failure modes.</td>
</tr>
<tr>
<td>Step 6b.3</td>
<td>Using assumptions, judgement and knowledge of user identify dominant failure modes, for example, by use of search algorithms. Also, decide on failure criteria to be adopted.</td>
</tr>
</tbody>
</table>
Step 6b.4 Build up the structural system to be studied from a number of series and parallel sub-systems.

Step 6b.5 Using assumptions, judgement and knowledge of user identify what factors need to be assessed, relevant to the focus of the study.

Step 6b.6 Using assumptions, judgement and knowledge of user perform component reliability analysis. Determine associated uncertainty.

Step 6b.7 Calculate reliability of the system and derive sensitivity measures.

Step 6b.8 Obtain a measure of the reliability of the structure. Present understand and interpret the results.

Step 6b.9 Perform comparison of reliability with pre-defined targets and acceptance criteria.

Step 6b.10 Determine uncertainty associated with the reliability measure derived.

Output 6b A measure of the reliability of the structure (e.g. probability of failure and reliability index), which has been compared to targets and acceptance criteria, with assessment of associated uncertainty.

<table>
<thead>
<tr>
<th>Table 75: Stage 6b: Reliability analysis using ‘system’ based approach</th>
</tr>
</thead>
</table>

7.3 Benefits and potential applications of the framework

The perceived benefits and potential applications of the framework have been identified and are examined in more detail in the following sections. They are listed as follows:

- Moving towards more consistent ‘true’ reliability
- Improved preparation
- Improved consistency
- Communication tool
- Application tool
- Management tool
- Quality assurance tool
- Education or training tool

7.3.1 Moving towards “true” reliability

To move towards “true” reliability it is first necessary to identify where uncertainty is introduced and secondly to study the uncertainty so as to find ways in which to reduce it significantly. The framework developed herein addresses the first part of the problem, by clearly and concisely presenting the steps required to perform a structural reliability assessment, which precisely identifies those steps which lead to the inclusion of uncertainty.

7.3.2 Improved preparation

The use of the framework would make it easier to see precisely what information is required before an assessment is started. The framework could be easily “filtered” to allow a list of all inputs to be produced.
7.3.3 Improved consistency

If all documentary outputs identified in the framework were produced, a full set of reports would be produced which show the assumptions and values used at a particular step during the assessment process. This would also have the benefit of encouraging a more consistent approach to assessments. Similar assumptions could be made for future assessments and the effect of changes in assumptions could be easily studied and perhaps quantified. The continued use of the framework would lead to improved repeatability and uniformity of assessments.

7.3.4 Communication tool

The framework has been produced in a format that lends itself to provision of guidelines, which will assist competent reliability engineers to perform a proficient reliability assessment. The format also enables the framework to be used as a “register”, so that an engineer can check whether activities have been undertaken correctly, in the right sequence.

7.3.5 Application tool

The framework was developed to identify each activity necessary to perform a full reliability assessment. It is envisaged as an application tool for performing reliability assessments, as any competent reliability engineer should be able to follow the steps that are shown on the flowchart and described within the corresponding tables.

7.3.6 Management tool

The framework could be a management tool - to aid project planning, implementation and checking. The relevant timings and resource allocations needed for each step could be anticipated and predicted in advance of the process being undertaken.

7.3.7 Quality assurance tool

The framework would allow a simplistic and traceable QA to be developed and performed on the reliability assessment process as a whole. QA checkers would be required to examine each step performed, and identify those areas where either the procedure has not been followed precisely, or where individual activities have been carried out with errors.

7.3.8 Education or training tool

The generic framework shows the step by step process necessary for the adequate performing of a structural reliability assessment. It should therefore eliminate some of the “mystery” surrounding the field of reliability analysis, and improve understanding of each of the stages within it. It should enhance comprehension of the process by showing in a clear, concise and visual presentation what steps are required. The provision of a list of the
corresponding references for each stage of the framework allows users to trace the development and reasons behind steps within the framework in context.

7.3.9 Potential usefulness of framework within project lifecycle

The perceived potential usefulness and benefits of applying the framework at the different phases of a typical offshore development project are summarised in Figure 95. The indications of benefits are arbitrary and to some extent subjective, but give an overall indication of the application of the framework.

The usefulness has been perceived in two ways: firstly, in terms of providing a more consistent approach to reliability assessments, and secondly, in terms of the potential optimisation and cost benefits of using the framework at different stages of an offshore platform project. An overall project has been divided into nine individual stages, which encompass a project starting from a conceptual study and concluding with decommissioning of the structure.

<table>
<thead>
<tr>
<th>Potential usefulness</th>
<th>Concept study</th>
<th>Feasibility study</th>
<th>Detailed design</th>
<th>Installation phase</th>
<th>Operations phase</th>
<th>Re-ass e.g. after damage</th>
<th>Re-ass e.g. after inspection</th>
<th>Re-ass e.g. change to deck loads or envt info.</th>
<th>Extension of original design-life</th>
<th>Decommissioning</th>
</tr>
</thead>
<tbody>
<tr>
<td>Low</td>
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<td></td>
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<tr>
<td>Medium</td>
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<tr>
<td>High</td>
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</tbody>
</table>

Figure 95: Representation of the potential usefulness of the framework at each project phase (Shaded areas represent perceived relevance)

7.4 Concluding remarks

The aim of this chapter was to present and describe the final framework that has been developed within this research programme. A generic framework has been developed for structural reliability assessment, which identifies the main steps required in order to carry out a system reliability assessment. The key technical and philosophical issues have been identified and linked together in a clear manner in the form of a series of flowcharts. These flowcharts present the framework in a format that is concise and rational. This format allows the relations between all of the issues raised to be seen at a glance.

As discussed in Chapter 3, the framework that has been presented is specifically applicable to fixed steel jacket-type offshore structures. It is focused on addressing the issues relevant to both the design of new installations, as well as the re-assessment of existing structures.
A number of significant changes have been made to the framework, in light of the research carried out as described in Chapters 4 to 6. The initial framework presented in Chapter 3 has been improved and expanded in order to address the key technical issues in more detail. The areas where uncertainty is likely to be introduced during a structural reliability assessment have now been clarified, and tasks that are potentially sensitive to the overall results have also been included in more detail. Due to this increase in detail it was necessary to divide the framework into manageable sections. This has meant that there are now four levels to the overall framework. The first is the top-level generic framework as shown in Figure 81, which has then to be studied in more detail in the second level as shown in Figure 82. This framework has a new symbol that signifies where there is a sub-framework. Four such frameworks have been developed. Figure 83 shows the sub-framework for the tasks that involve the assessment of platform conditions. Figure 85 then shows the sub-framework for the environmental loading assessment stage.

The foundation assessment process has then been studied in further detail and the next two figures show the tasks for the foundation assessment process. Figure 86 to Figure 88 relate to the assessment of pile foundations in sand and in clay soils.

The reliability assessment stage is then also presented in more detail with two key flowcharts used to represent a reliability ‘component’ or a ‘system’ assessment in Figure 89 and Figure 94 and respectively. The flowchart used to present the ‘component’ based approach has then been examined further, and four flowcharts are used to represent the actions required for the ‘minimal’ analysis (Figure 90), response surface (Figure 91 and Figure 92), and numerical simulation (Figure 93) approaches.

The tabular format as initially proposed in the preliminary development described in Chapter 3, has been re-examined in the light of the results and findings from the studies presented in Chapters 4 to 6. The outline tabular framework has been updated to incorporate changes and improvements, as shown in Table 64. The detailed individual tabular frameworks format has been developed. A series of individual tabular frameworks have been presented in Table 65 to Table 75. It is envisaged that these tables will be studied alongside the flowchart frameworks. The arbitrary scales proposed for representing the relative level of uncertainty, sensitivity, complexity and operator competence have not been updated. The scales proposed were based purely on a subjective approach to these issues. The potential benefits are debatable and prone to a certain degree of approximation. Further work could be focused on addressing these issues in an alternative manner, if required.
The framework presented herein has been assessed in terms of its potential usefulness and the perceived benefits that can be derived from its application. If the framework is applied to the performance of structural reliability assessments, the main benefit will be a move towards more consistent reliability assessments in the future. The main aspects of each of the benefits and potential applications considered for the reliability assessment frameworks developed herein have been summarised.
CHAPTER 8.

CONCLUSIONS

8.1 Introduction

The objectives of this research have been described in Chapter 1, with the main aim being to develop a generic framework, which will set the basis for achieving more consistent system reliability assessments. This key aim has largely been achieved. The specific conclusions and observations derived in each area considered have been given in detail at the end of each chapter. As such, this chapter aims to reiterate the main discussion points and to clarify the main conclusions of the research work described in this thesis.

The preceding chapters have included detailed discussion of results and findings of the individual parametric and sensitivity studies. To reiterate, the overall aim of this research has been to move towards more consistent reliability assessment of offshore structures. A framework has been developed which identifies the key technical and philosophical issues that need to be considered in structural system reliability analysis. Key areas where uncertainty is introduced have been studied in detail and individual conclusions regarding specific aspects have been drawn.

A model of the production platform Leman AP has been used within 3D non-linear finite element analysis to investigate the sensitivity of system strengths and reliability to a number of resistance parameters. This has been studied within the context of a number of different reliability methods that are currently used in the offshore industry. The combined deterministic and probabilistic studies performed have enabled a detailed framework to be developed. The resulting framework provides a simple and effective 'communication tool' for use by engineers to converge towards more consistent reliability predictions.

The results presented and the corresponding discussions in this thesis are based on the study of a specific platform structure under, predominantly, one wave approach direction. The conclusions drawn from this research are therefore structure dependent and should not necessarily be considered to be representative of other structures in different locations.
although the overall characteristics observed are considered to be applicable to other fixed offshore structures. The specific details of the trends should be interpreted with caution outside the limits of applicability. Further work is needed in order to establish whether the trends observed are similar for other structures and other foundation scenarios.

8.2 Review of system reliability assessment

The current status of system reliability assessment in the offshore industry was described in chapter 2. 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 chapter covered an introduction to the problems associated with reliability assessment, briefly described generic reliability issues and also the reasons behind uncertainty and sensitivity. Sections then followed which introduced all the major aspects of reliability analysis and a number of case studies were then reported, along with an investigation into different reliability approaches currently used offshore.

The key issues that were identified in the review study were segregated into those that were generic and those that were applicable to the specific example of fixed offshore structures. A summary of the key stages in reliability analysis, along with related technical and philosophical issues that were identified in the review study was presented. These issues formed the basis of the subsequent framework development.

8.2.1 Summary of generic issues relating to offshore structures

The main qualitative aspects that have been identified relating to offshore structures, discussed in Chapter 2, are briefly summarised in the following sections:

Probability of failure

- Reliability involves dealing with events whose occurrence at any time cannot be predicted where the probability is expressed by likelihood of the event occurring:
- An absolute measure of reliability is only obtained when physical uncertainty dominates over model prediction uncertainty, assuming that the physical uncertainty can be represented accurately, thus minimising statistical uncertainty.
- Reliability analysis for offshore structures involves the generation of directional long-term statistics of extreme load, calculation of ultimate strength, an estimation of uncertainty in the structural strength and then calculation of the probability of failure.

Uncertainties and sensitivity

- 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 which is more critical when the activity has high uncertainty in methodology or is highly sensitive to the overall reliability result.

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.
- Reliability results used to be taken as an indication of notional reliability, but changes in modelling/software have helped to minimise 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 - any comparisons undertaken must be strictly on a like-for-like basis.
- To improve consistency of results increased awareness of uncertainties/sensitivities at each step of a reliability analysis is needed.
- Development of a framework to identify main steps will go towards improving overall structural reliability and consistency.

Competence and guidance for users

- There is a need to move towards guidelines for a more rational approach.
- Need to reduce/better quantify modelling uncertainty, and consider improved means of incorporating it into reliability analysis.

Evaluation 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 have identified the need for some kind of framework or general procedure to assess offshore platforms with a variety of failure modes, with a more rational and consistent approach.

8.2.2 Summary of issues specific to fixed steel platform type offshore structures

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 with the inertia and marine growth coefficients modelled as deterministic. Uncertainty in drag coefficient cannot be ignored - COV can be in the order of ~20%.

Loading uncertainties

- Loading variables can account for up to ~95% of total uncertainty (when foundations are ignored). Need more data to develop joint probability distribution of 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%.

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.
- Where foundations are included in the analysis the response uncertainties require assessment and can be of the same order of uncertainty as the loading.
- A degree of uncertainty exists about validity of foundation modelling and of data used for soil parameters.
- The foundation capacity derived by the API method has a COV ~32%, and by the IC method has a COV ~22% for piles in sands.
- Yield strength is also a key factor for the resistance, and can be represented with a COV in the order of 5% to 12%.

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 choice of probability distribution types.
- It can be derived from the ratio between true quantity and 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.
- Modelling uncertainty in piled foundations in sand assessed using the API method has a bias of 0.84 and a standard deviation of 0.56. The IC method has a bias is 0.97 and a standard deviation of 0.28.

Environmental extremes

Conventional treatment of waves, current and wind forces is 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%.

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. More extreme wave approaches, combined with more susceptible structural configurations can lead to an overestimation of ultimate base shear in the order of ~2.

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. 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. The reserve strength ratio will be expected to be in the order of ~2.

Airgap

Need to improve understanding of the issues surrounding the derivation of the airgap. In the past, airgap had to be greater than 1.5m. Now, it has been defined with a probability of occurrence of less than say ~10^6.

8.3 Structural system reliability framework development

The initial development of a generic framework for system reliability assessment, concentrating on the main steps in a reliability assessment and the key related technical and philosophical issues, was described in Chapter 3. These issues were linked together in a flowchart arrangement, in order to present a rational and concise framework. A generic framework was developed for use for both design/new and existing/reassessment of structures and which was applicable to both fixed and floater types of installation. This provides a concise and succinct method of presentation, although it does not allow in-depth detail of the steps to be presented. It was for this reason that the outline tabular framework was also developed.

In order to study the generic framework in more detail and to break down each stage into individual activity steps, a specific example had to be adopted before a more detailed framework could be presented. The specific example of the design of a fixed offshore
platform within the North Sea was adopted and developed further. A similar development to another more detailed form can be undertaken for different offshore applications.

The framework developed at this stage includes all the main steps identified through the review study. A top-level framework indicated the main elements of the generic framework starting with the inputs required relating to platform description, foundation and environmental parameters. The next eight stages dealt with the detailed steps required to undertake a reliability assessment and include an assessment of the fixing of the structure, modelling of the structure, foundation capacity derivation, load derivation, system analysis model derivation, ultimate capacity derivation and then the reliability analysis. The final stage was the collation of outputs of each of the stages, resulting in a measure of reliability and comparisons with reliability targets if required.

Examples were extracted of those areas that could be identified from the generic framework where external constraints were likely to impinge. These were as follows:

- Stage 2. Modelling of structure: Decision as to what software package to use
- Stage 4. System analysis model derivation: Complete structural analysis using various software options
- Stage 4. Ultimate capacity derivation: Decision as to which methodology to adopt.

The generic framework was studied with a view to focusing the subsequent phases of the research. Areas where future work was needed to improve the methods and to converge towards more consistent reliability predictions were identified. Such areas were where significant uncertainty existed or where results were highly sensitive to parameters or processes. These were discussed in detail in chapter 3 and are listed here:

- determination of environmental parameters’ statistical distributions
- determination of foundation capacity and its distribution
- decision on modelling method of foundations
- performance of pushover analysis and determination of ultimate capacity
- determination of strength distribution (member or structure) and
- derivation of probability of failure.

Based on the above, the following conclusions can be made:

- A number of important areas where further work could be focused have been identified, of which, there are several outstanding issues that have not be addressed
in detail before. These areas were identified as the determination of foundation capacity and its distribution and the modelling method of foundations.

- When the foundations are ignored in an analysis, then the yield strength had previously been found to be a dominant factor on the resistance side.

- No investigations to date had looked into the effects of foundation capacity and stiffness, alone or in conjunction with the effect of changing the yield strength.

- Significant differences existed in the way that foundation capacity was assessed. This has a significant role in the prediction of the resistance of the structure in order to perform a structural reliability assessment.

- The uncertainty of individual steps in the reliability analysis procedure was also an area identified for further work, including the specific approach or method adopted, modelling uncertainty, study of foundation effects and study of system effects.

8.4 Investigation into key parameters

It was noted that on the resistance side, the yield strength had previously been identified as a source of uncertainty, when foundations had been assumed to be fixed. However, in most cases, it can be inappropriate or unconservative to ignore foundations completely. The capacity and stiffness of foundations were therefore examined in detail, in conjunction with the yield strength, in order to assess their individual influences and possible interaction. The uncertainty of individual steps in the reliability analysis procedure was also an area identified for further work, including the specific approach or method adopted, modelling uncertainty, study of foundation effects and study of system effects. The variability in different methods such as search algorithms, pushover analysis assisted by simulations and simplified system reliability methods was studied and described in chapters 4 to 6.

The greater the increase in yield strength, the higher the peak load factor exhibited and conversely, the lower the yield strength, the lower the peak load value shown. It was found that linear equations gave a good fit to the data. As the yield strength was increased a proportional increase in the peak load factor was exhibited. An increase from −3 to +3 standard deviations produced an overall increase in the peak load factor of ~13%. The failure mode for those cases run with increased yield strength were found to be jacket dominated, and those case with decreased yield strength where found to exhibit a mixed mode failure.

A detailed investigation was undertaken to assess the effect of changes in foundation capacity on the peak load factor exhibited. For those runs performed which had a decrease
in the overall foundation capacity, the trend could be represented by a second order polynomial. The runs with an increase in the pile spring deflections exhibited an increase in the overall stiffness of the load-deflection behaviour of the platform. In contrast, those runs with a decrease in the pile spring deflections exhibited a decrease in the overall stiffness of the load-deflection behaviour.

Further investigation revealed that there were three distinct regions along the curve in terms of dominant failure mode. The initial region, where there is a linear relationship between foundation capacity and peak load factor, was found to be foundation dominated. The final region where there is a ‘plateau’ on the trendline, where increases in foundation capacity were not reflected by increases in peak load factor, was found to be jacket dominated. The region that exhibits a curve was found to produce a mixed mode failure, consisting of both foundation and jacket failure.

Based on the above, the following conclusions can be made:

- For the structure examined here, the stiffness of the foundations had a minimal effect on the overall ultimate capacity prediction - foundation capacity dominated.
- Changes to the foundation capacity exhibited three regions of failure mode.
- At low foundation capacity, a region of foundation-dominated failure was exhibited. In this region it was found that yield strength changes did not affect the ultimate capacity, and that foundation capacity dominated.
- At high foundation capacities, jacket-dominated failure modes were exhibited. In this region it was found that foundation capacity changes did not affect the ultimate capacity, and that yield strength dominated.
- An intermediate region of ‘mixed’ mode of both foundation and jacket failures was also exhibited. Both yield strength and foundation capacity affected the ultimate capacity in this region.

8.5 Investigation into key parts of the process – assessment of foundations

Investigations into the assessment of foundation capacity were undertaken in detail. This focused on three key areas where the effect on the ultimate capacity of the structure was studied for different soil types, soil profiles and capacity assessment methods. Additional aspects included in the IC assessment method were also studied including analysis of the effects of cyclic loading and ageing, on both a jacket-dominated failure scenario and a mixed mode failure scenario.
A study of four different synthetic soil profiles was carried out. When the four profiles were adopted and the API assessment method used to predict foundation axial capacity, it was found that there was very little change in the predicted capacity. However, when the IC method was used, significant differences in the capacity were noted. This was because the API method was insensitive to any changes in soil profile once the soil has been classified as ‘very dense’ whereas the IC method was sensitive to soil profile changes. Comparisons were made between the API and IC assessment predictions of axial foundation capacity, based on the same soil profile. It was found that under compression loading the results were similar, but with API predicting marginally larger capacities.

The effects of cyclic loading and ageing were assessed for piles in sand when using the IC assessment method. For a jacket-dominated failure mode it was found that negligible changes to the ultimate capacity of the structure were exhibited. This was due to the fact that the foundations were adequate to ensure that failure still occurred in the jacket, despite the changes caused by cyclic loading or ageing. However, when the structure with a mixed mode failure was studied, the detrimental effect of cyclic loading, with no allowance for ageing, changed the failure mode to a foundation dominated scenario, and a corresponding decrease in the ultimate capacity of the structure was predicted. When the positive effect of ageing was applied with no cyclic loading an increase in peak load was exhibited. This case exhibited a jacket-dominated failure mode.

When the foundation stiffness was studied, it was found that negligible changes to the peak load were exhibited with change in stiffness and it was therefore concluded that foundation capacity dominated the foundation characteristics and their effects on the structural system behaviour.

Part of the research described has focused on the issues surrounding the improvement of the assessment and characterisation of foundations. The issues specific to piles in sand have been studied in detail, with the aid of non-linear finite element analysis. The following conclusions may therefore be made:

- The degradation effect of cyclic loading and the beneficial effect of ageing have negligible overall influence on the ultimate capacity in a jacket-dominated failure scenario.
- For mixed mode or foundation-dominated failure the degradation effects of cyclic loading are potentially greater than the beneficial effects of ageing.
• The API assessment method is insensitive to changes in soil density, once a 'very dense' soil has been assigned.

• In contrast, the newer IC method allows for increasing capacity with increasing soil density. In order to move towards an accurate prediction of foundation capacity, this method is therefore recommended for piled structures in sand soils.

8.6 Reliability assessment methods

An investigation into the main reliability assessment methods used within the offshore industry has been undertaken. Three key methods have been studied in detail: the minimal analysis approach, the response surface approach and the system based approach. The three reliability approaches studied are fundamentally different in the way that the loading and resistance are represented in the reliability calculation.

The minimal analysis approach is computationally simple and requires only a small number of FE runs to be performed in order to derive a failure surface compass-diagram. If the distributions are both lognormally represented then standard equations can be used to derive the reliability index. This method is really only suited to cases characterised by low resistance uncertainty when compared to the loading uncertainty. If this method is used within the limits of its applicability, then it has been found to give sufficiently accurate results, provided that the COVs for the two variables are suitably calibrated. While a simple loading model was used in this investigation, more accurate models can be used in conjunction with the minimal analysis approach. Such models would be derived from a more rigorous processing of the environmental data which would significantly improve the accuracy of the results.

The response surface method was developed within this project to incorporate the foundation uncertainty and the effects on system reliability. This method allows a simple or complex loading distribution to interact with a complex surface representation of the resistance. A simple loading distribution will be based on experience, whereas a complex loading distribution will be developed from a rigorous and detailed processing of the raw environmental data. Either of these loading representations can be used in conjunction with the minimal analysis approach or the response surface technique.

The response surface approach provides efficient methods for keeping the number of pushover runs to a manageable number, if methods such as the central composite design are applied. This method allows the use of advanced structural analysis in combination with
system reliability methods. It aims to capture the key elements of the resistance within a suitable range. However, problems can be associated with the representation of complex trends and characteristics. Information about the location of the design point will be an output from the reliability analysis. If this point falls outside the original surface, additional runs are required in order to ensure that the surface is sufficiently accurate in the region of the design point. As with other methods, changes to the reliability index and probability of failure will be caused by changes to the distribution of individual parameters comprising the response surface and care should be taken that the most accurate representation of these parameters is used. This method enables the use of advanced structural analysis techniques to be combined with system reliability methods to derive an improved representation of the global resistance of the structure.

In the system analysis approach is a rigorous approach in which search algorithms are used to identify the most dominant failure paths. However, it does involve a number of approximations and it is possible that a dominant failure mode could be overlooked. It is important to perform such an analysis with previous knowledge obtained from a simplified or deterministic analysis. This method incorporates the most detailed representation of the environmental loading on the structure out of the three methods examined here. It would, however, be possible to combine more detailed loading models with the minimal and response approaches.

The following conclusions may be made based on the above:

- Investigations into different reliability methodologies used in the offshore industry have been undertaken in detail. The background to the three main approaches has been presented along with details of the key features.
- Detailed investigations into the different methods used to assess reliability have shown that there are significant differences in the approaches used in the offshore industry.
- In order to move towards more consistent reliability, the best possible representation of the environmental loading is required. For the loading, it is concluded that this would either be in terms of representing the individual parameters that make up the environmental model, or by derivation of ‘simple’ loading model based on elaborate processing of environmental data. However, one disadvantage of this approach is that it requires significantly more data and requires notably more analysis work in order to derive the appropriate distribution and COVs for each parameter. Such data may not be available and the analysis may thus be forced to use the simple distribution approach.
• On the resistance side, the method in which the resistance or response of the structure is represented differs significantly according to which approach is used. The probabilistic description for the strength and stiffness of structural members depends upon the probabilistic description of the members and joints, such as the cross-sectional dimensions and material characteristics.

• The resistance can be treated with varying degrees of complexity. In order to move towards more consistent reliability the best possible representation of the resistance is required. One such approach could be to use the response surface technique. This method lends itself to investigations where different resistance parameters need to be taken into account in the reliability assessment.

• From the experience gained by using the response surface technique, a number of issues arose which could be considered when undertaking future RST work. Care is needed when choosing the equation to represent the surface, deciding on the distribution of the various resistance parameters.

In order to make decisions concerning the level of detail required for a system reliability assessment on an offshore structure, it was found that a suitable starting point is to examine the dominant failure mode. To do this, it was found that merely examining the deflected shape of the structure at peak load did not necessarily provide a conclusive assessment of the failure mode. Two other techniques were also used to provide an insight into the mode of failure. These were the examination of the location and number of plastic hinges, and an assessment of the pile utilisation, also known as soil mobilisation. The combined information could be examined in the light of the load-deflection characteristics of the structure in order to make an assessment of the failure scenario. Although this approach is not unique to this research, the FE post-processing required to perform these tasks was developed within this study.

Based on these findings, some preliminary work has been undertaken in order to present a new hypothesis. The application of this approach is aimed to aid the engineer in deciding when more detailed foundation and reliability assessments are required, and when detailed foundation assessments would be superfluous. Further work is required in order to develop this theory and to enable guidance as to the range of foundation capacity that is needed to be assessed before the reliability analysis stage.
8.7 Final structural system reliability framework

The aim of chapter 7 was to present and describe the final framework that was developed within this research. A generic framework was developed for structural reliability assessment, which identifies the main steps required in order to carry out a system reliability analysis. The key technical and philosophical issues have been identified and linked together in a clear manner in the form of a series of flowcharts.

A number of significant changes were made to the framework, in light of the research carried out as described in Chapters 4 to 6. The initial framework was improved and expanded in order to address the key philosophical and technical issues in more detail. The areas where uncertainty is likely to be introduced during a structural reliability assessment have now been clarified, and tasks that are potentially sensitive to the overall results have also been included in more detail. Due to this increase in detail it was necessary to divide the framework into manageable sections. This has meant that there are now four levels to the overall framework. The first is the top-level generic framework, which was then studied in more detail. Four such frameworks were developed. Sub-frameworks were then created for those key areas that were developed at the highest level of detail.

The tabular format initially proposed in the preliminary development was re-examined in the light of the results and findings from the studies presented in Chapters 4 to 6. The outline tabular framework was updated to incorporate changes and improvements. The reference tabular framework has also been updated. The detailed individual tabular framework format has been developed and it is envisaged that these tables will be studied alongside the flowchart frameworks.

The perceived benefits and potential applications of the framework were identified and have been discussed in detail in chapter 7. They are as follows:

- moving towards more consistent reliability
- improved preparation
- improved consistency
- use as a communication tool
- use as an application tool
- use as a management tool
- use as a quality assurance tool or
- use as an education or training tool.
The following conclusions may be made regarding the framework presented:

- A unique framework has been developed and presented herein.
- It provides a clear and concise presentation of the technical and philosophical issues and their interrelations, which constitute a structural system reliability analysis.
- The perceived benefits and potential applications of the framework provide an aid to move towards more consistent reliability predictions.
- The framework provides an effective ‘communication tool’, which can assist competent reliability engineers to perform a proficient reliability assessment with a more consistent and rationalised approach.
- Areas where uncertainties are introduced have been identified, and detailed studies have been undertaken to improve understanding of some of the key uncertainty issues.
- The framework presented herein has been assessed in terms of its potential usefulness and the perceived benefits that can be derived from its application.
- It is envisaged that the framework will lend itself to both the design stage and reassessment.

8.8 **Original work and contribution to knowledge**

The research presented here has provided valuable information and a unique insight into the following key areas:

- Development of a structural system reliability framework for fixed offshore structures for more consistent reliability assessments.
- Presentation of the current status of structural reliability analysis for fixed offshore platforms.
- Identification of areas where difficulties still remain with this analysis which prevent more consistent reliability predictions being made.
- Assessment of the relative significance of and sensitivity to the main parameters in a reliability analysis.
- Details for assessing the significance of various parameters in a reliability analysis given certain deterministic aspects. In particular, details on foundation assessment methods and parameters.
- The response surface technique has been applied in the system reliability assessment of a fixed offshore platform. This is the first time that a clear and comprehensive account of the use of this approach has been made, including taking into account foundation reliability which has not been incorporated previously.
Some guidelines on the application of the response surface technique for the reliability assessment of fixed offshore platforms has also been introduced.

Information and guidance on developing appropriate and consistent reliability methodologies, in particular, in the form of a structural system reliability framework.

Recommendations on areas that require further research to improve current methods and to facilitate a convergence towards consistent reliability predictions.

8.9 Areas for further work

The following section summarises those areas where further work would be beneficial in converging further towards improved understanding and more consistent reliability assessments.

Within the framework development various sources of uncertainty have been identified and their relative significance has been examined. Further examination is needed on how areas of uncertainty are currently modelled, how modelling uncertainty can be improved and identification of advantages and disadvantages of the different methods.

This research has addressed some of the key issues relating to foundation uncertainty and the sensitivities involved within the context of system reliability analysis of fixed jacket structures. Studies have been undertaken to assess the effects and trends that may be expected from different soil types and assessment methods (API and IC) and a detailed study was undertaken for piled foundations in sands. Further work would be recommended to refine the methods developed for sands and to address similar issues in clay foundations where different trends may be encountered. Furthermore, a new ISO code is currently under development and it would be appropriate to perform similar studies with the new procedure and to make detailed comparisons in order to assess the differences between the existing API and IC approaches and the new recommendations.

In this research, the simplified environmental approach of using the 50-year wave, the 50-year wind and the 50-year current occurring simultaneously, acting in the same direction, was sufficient to study the resistance aspects described. For a typical jacket structure, this will lead to the derivation of a 50 year ‘design’ load which is typically more severe than the ‘true’ 50 year load. The traditional practice is conservative in two ways: extremes do not necessarily occur simultaneously and extremes will not necessarily combine in the worst
possible way. If required, future work could apply a method which accounts for the joint probability of occurrence of extreme events such as the 50-year and 100-year events.

Within this study, for the analyses undertaken, no checks were performed on the utilisation of tubular joints as required according to API RP2A. This would normally be performed as part of a detailed assessment of an installation. It should be noted that if joints are correctly designed and are not affected by weld defects or damage, then it is generally considered that members will fail before the joints, and hence the results obtained in this study should not be adversely affected by this. However, further work could incorporate joint utilisation checks if required.

In order to give a wider view and understanding it would be useful to study alternative failure criteria. One such example could be to place a limit on the global deformation that would exceed equipment limits on the deck. In the case of piled foundations it would also be useful to investigate the occurrence of pile pullout, as it has not been feasible to study this within the scope of the current research.

Since the withdrawal of the HSE guidance notes, the offshore industry has become more aware of the significance of the airgap issue. A number of initiatives are currently underway in order to improve understanding of the issues surrounding the derivation of the airgap, and to potentially move towards an industry accepted performance standard. Further work is needed on how potential erosion of the airgap can affect the reliability of a fixed offshore structure.

The framework developed and presented in this thesis has been based on fixed steel platforms and on their ability to withstand extreme weather. Further work could be carried out to extend the framework to other types of offshore structures, or to address additional major hazards such as ship impact, fire and blast, or fatigue damage scenarios.
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