Ultimate Strength Performance of Offshore Structural Framing
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HEALTH & SAFETY EXECUTIVE
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ULTIMATE STRENGTH PERFORMANCE
OF OFFSHORE STRUCTURAL FRAMING

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SUMMARY

This review of the use of nonlinear ultimate strength (pushover) analyses in determining performance measures of tubular framed structures has been prepared for the Health and Safety Executive (HSE) by BOMEL Limited. An in-house state-of-the-art package (SAFJAC), developed over the last decade in parallel with experimental investigations, was used to analyse a variety of frames representative of platforms in the UK Sector of the North Sea. Ultimate strength analysis of offshore structures has increasingly become used to assess the integrity of various design configurations against extreme and accidental loading. It has also been used to determine the safety of existing offshore structures in view of updated environmental and structural data or changing operational requirements. It is now also being adopted to determine the criticality of members within the structural system and to assess various inspection and repair schemes.

The output from nonlinear pushover analyses has frequently focused on the ultimate strength expressed in terms of a reserve strength ratio (RSR) comparing the ultimate strength to the 'design' level (e.g. 100 year environmental loads). The sensitivity of RSR to imperfections, defects or damage or to variations in assumed loading pattern or intensity may not be recognised. However the ability of the structural system to deal with these uncertainties, its robustness, may be crucial to surviving extreme or accidental events. With the increasing use, and range of application, of ultimate strength analyses, it becomes important to understand how various performance measures may provide insights into the strength, redundancy and ductility of steel platform jackets. The effect of alternative framing configurations on these measures should also be examined.

This report draws together the results from published investigations and identifies key sources contributing to uncertainties and key factors contributing to reserve strength. Deterministic sensitivity-independent, deterministic sensitivity-dependent and reliability-based performance measures are reviewed, and the advantages and disadvantages of each are identified. Differences in the definition of reserve strength are noted, underlining difficulties in drawing comparisons between structures.

Next, a variety of two-dimensional frames representative of UK Sector North Sea jackets was analysed using SAFJAC. It is shown that bracing configurations are important influences. Jacket examples are cited where first failure precipitates global collapse. Other structures are shown to be sufficiently redundant to sustain loads well in excess of the design value, with collapse occurring only after a sequence of component failures under increasing load. The effects of initial imperfections, material behaviour, joint flexibility and distribution of still water to hydrodynamic loading on the ultimate strength and post ultimate behaviour of jackets are also examined.

A variety of performance measures, reflecting different measures of ultimate and post-ultimate behaviour, was developed and validated in this study. This review also includes guidelines for carrying out sensitivity studies.
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1. INTRODUCTION

1.1 PREAMBLE

The past decade has witnessed a significant increase in the use of ultimate strength analysis to assess the reserve strength of steel offshore jacket structures and to determine the redundancy and the post ultimate behaviour of the structural system. This activity has been partly motivated by the following developments:

- With the increasing interest in taking advantage of the system strength beyond first component failure to resist extreme loading events particularly for ageing structures, it is important to understand how the various parameters are critical to strength and how variations in these parameters may affect strength.
- Trends for lighter, liftible jackets where the omission of members with low elastic utilisations may save weight, but also reduces the capacity for redistribution along alternative load paths.
- Development of new concepts for deeper waters and more efficient jacket configurations where it is necessary to ensure that levels of reserve strength and safety embodied within older designs are maintained.
- Commercial pressures to target inspection and maintenance schemes more effectively at components critical to system integrity.
- Ability of modern software packages to model salient features of steel offshore jacket structures, and the availability of powerful computer hardware which makes the use of modern software packages feasible (in terms of CPU time and hard-disc storage capacity).

However, nonlinear computer analyses generate a vast amount of data and therefore a rational method for assessing the accuracy of the computer model and the validity of the results is required. Indeed, the need to provide engineers with guidance on carrying out ultimate strength analyses and interpreting the results of such analyses is recognised by the offshore oil and gas industry which has sponsored a joint industry project to produce best practice guidelines for the use of nonlinear analysis methods for offshore structures (Ultiguide, 1998). Furthermore, there are various measures for assessing the system performance at and beyond its ultimate capacity. These various performance measures must be carefully examined and their advantages and limitations highlighted. To this end, the original review carried out by BOMEL in 1996 (OTH 92 365) will be modified and extended.

1.2 PROJECT OBJECTIVES

The objectives of this study were as follows:

- To compare the ultimate capacity and post ultimate behaviour of offshore structural framing
- To investigate the sensitivity of the system strength and ductility to the strength of individual structural components
- To provide guidance to the HSE on the relative performance of different framing arrangements in order to assist in the evaluation of offshore platforms under extreme loading conditions
• To provide insight to HSE engineers on the performance of nonlinear platform analysis and the SAFJAC software in particular.

To achieve the above objectives, the following six technical tasks were undertaken:

• Task 1 - Development of design / analysis premise
• Task 2 - Selection of framing configurations, covering 4, 6 and 8 legged structures for analysis
• Task 3 - Design of each case
• Task 4 - Ultimate load analysis of each case using SAFJAC
• Task 5 - Sensitivity study on input parameters for selected cases
• Task 6 - Comparison of results and development of performance measures

The work performed and the findings are described in this report.

1.3 LAYOUT OF REPORT

The remainder of this report is divided into ten sections. In Section 2 the scope of work and the objectives of this study are presented. Section 3 discusses the methodology adopted in the selection of the frames, the loading acting on the jackets and the design methodology.

Section 4 presents the structural models used in the pushover analyses. The results of the base analyses and the sensitivity study are presented in Sections 5 and 6. Section 7 provides a discussion of the results with particular emphasis on framing systems and performance measures. Finally, conclusions are drawn in Section 8.
2. SCOPE OF WORK

2.1 INTRODUCTION

Ultimate strength analyses are often carried out to determine the reserve and residual strength, the redundancy and mode of failure of the jacket. However, there is no consensus regarding which performance measures should be used in determining the above parameters. Indeed, a variety of performance measures are used by the industry. These, amongst others, will be reviewed in this section. To this end the extensive study carried out by BOMEL in 1996 (OTH 92 365) will be modified and extended. A number of these performance measures will then be selected and tested for a range of structural frames.

2.2 REVIEW OF PERFORMANCE MEASURES

2.2.1 Reserve strength

Concepts of reserve strength were introduced in relation to seismic assessment where Blume's 'Reserve Energy Technique' (Blume, 1960) defines the reserve capacity B, as:

\[ B = \sqrt{\frac{\text{Energy Capacity}}{\text{Energy Demand}}} \]  

Eqn 2.1

Reserve strength is now more commonly defined as the ability of a structure to sustain loads in excess of the design value. The Reserve Strength Ratio (RSR) (eg. Titus and Banon, 1988) may be defined as:

\[ \text{RSR} = \frac{\text{Ultimate Platform Resistance}}{\text{Design Load}} \]  

Eqn 2.2

In a recent HSE study (OTO 97 046), it was suggested that an approximate value for the reserve strength may be obtained by examining the proportion of design load that is environmental in nature. Assuming a global factor of safety is being applied to the combined dead and live loads, by removing the factor of safety and assuming the associated margin is available for increased environmental loading an estimate of the reserve strength may be obtained. Clearly the higher the proportion of the dead load to the environmental load in the original design, the lower the safety margin available for any increase in environmental loading. This simplified methodology was initially proposed by Bea and Mortazavi (1995) and later adopted in the above report as a conservative estimate of reserve strength. The above HSE report identified the available air gap as an additional parameter which must also be considered when assessing the probability of survival. The real sea state (and associated return period) which may cause collapse is higher than that usually used in ultimate strength analyses. This higher wave may breach the air gap and result in additional load being applied higher up the structure. Therefore, the mode of failure of the structure may change and, in particular, the deck legs may fail. The ISO guide (ISO 2394) defines the design load as the unfactored 100 year global environmental action, while the ultimate platform resistance is determined in terms of the unfactored global environmental load to cause collapse.

From the above discussion, it should be recognised that a structure with a lower reserve strength and a higher airgap may have a higher probability of surviving an extreme storm loading than another structure with a higher reserve strength and lower airgap.
The various procedures for determining the design environmental load are discussed in detail in Section 2.2.3.

Other than the ratio of the ultimate platform resistance to the design load, RSR is also quoted as ratios of platform base shear or overturning moment. It should also be recognised that for a single platform there is a separate RSR for each load case or load combination. Indeed, the loading case which produces the highest component utilisation at the design load level or the highest overall base shear will not necessarily result in the lowest RSR.

Reserve strength should not be solely considered as overdesign of structures, rather it is required to cope with loads which have not been foreseen in the design process or loads which cannot be economically designed for an elastic basis (eg. seismic or accidental loads). It is therefore important to understand the sources of reserve strength inherent in traditional design procedures.

2.2.2 Sources of reserve strength

Reserve strength exists at the component level to allow for uncertainties in both the resistance of the component and the loading to which it is subjected. Based on statistical data, characteristic values are adopted to ensure that the probability of failure is acceptable. Beyond that, safety factors are applied to improve the certainty of survival and to allow for factors for which no statistical data are available (eg. for inaccuracies in structural analysis techniques). It is clear that the actual capacity of a component is likely to exceed the allowable loads for which it is designed.

At the system level, however, there are additional sources of reserve strength. The failure of one component may not limit the capacity of the structure as a whole, provided there is adequate ductility and redundancy such that loads can be redistributed. For highly redundant structures, a sequence of component failures may occur before the ultimate strength is reached.

The above factors are implicit sources of reserve strength which are not generally controlled or quantified in design. Similarly, conservatism embodied within codes, material yield strengths exceeding the minimum criteria specified, component limit states less onerous than ultimate strength and overdesign of non-structural requirements, may be considered as implicit sources of reserve.

By contrast, overdesign by exceeding minimum requirements or by conservative combinations of loads are sources of explicit reserve and can be controlled by the designer.

2.2.3 Design procedures

Concepts of reserve strength are inextricably linked to design criteria and it is therefore necessary that typical design procedures should be reviewed. For most existing structures, and some conventional jacket structures in the near future, working stress principles apply. From a working stress viewpoint, design loadings (eg. from a 100 year return period storm) are applied to the structure and the forces and moments in the components are compared with the 'allowable' values taken from the prevailing Codes of Practice, encompassing the appropriate factor of safety. Typical guidelines therefore address how the elements of a structure should be proportioned, but not how the assembled elements or structural system should perform.

The WSD acceptance criteria for components may be given by the following inequality:

\[ \frac{R}{F} \geq (D + E) \]  

Eqn 2.3
where

\[ R \] is the characteristic ultimate resistance or strength

\[ D \] is the stillwater loads

\[ E \] is the environmental loading due to wave, wind and current in the event of storm conditions

\[ F \] is the factor of safety which varies with component and loading mode. (In analysing storm conditions given by \( E \), a \( \sigma \) overestress is generally allowed. In the case of a tubular joint, for example, the normal safety factor of 1.7 is thereby reduced, so \( F = 1.7 / 1.33 = 1.28 \)).

Load and resistance factor (LRFD) limit state design codes are now in place and although these still focus primarily on component adequacy, in some instances they also contain explicit provisions for system behaviour. By contrast with the all encompassing safety factor, \( F \), in WSD, a range of partial factors, \( \gamma \), are adopted for LRFD to reflect the respective uncertainties on individual elements of loading and resistance.

\[ \gamma_M \cdot \gamma_S \cdot R > \gamma_D \cdot D + \gamma_E \cdot E \]  
Eqn 2.4

where

\[ \gamma_M \] is the material coefficient

\[ \gamma_S \] is the structural coefficient

\[ \gamma_D \] is the stillwater load coefficient

\[ \gamma_E \] is the environmental load coefficient

Comparing Equations 2.3 and 2.4, it can be seen that under WSD, a significant margin, corresponding to the safety factor \( F \), is required between the applied loads, \( D \) and \( E \), and the available resistance. By contrast, for an LRFD structure, the design loads are the factored loads, \( \gamma_D \cdot D \) and \( \gamma_E \cdot E \), thus the required margin between the resistance and design loads relates only to structural and modelling uncertainties contained in \( \gamma_M \) and \( \gamma_S \). Given this discrepancy, an RSR relating ultimate platform resistance to design load needs careful qualification to prevent confusion in identifying a target RSR.

In the context of this study, the WSD approach will be adopted in the design of the frames. However, for pushover analyses, the environmental loads are incremented while the stillwater loads are held constant.

### 2.2.4 Redundancy

Members which carry negligible load under elastic loading, can provide a significant contribution to maintaining overall resistance in the event of damage to other parts of the structure. For example, in the event of damage to diagonal braces, horizontal members at plan framing levels would be essential for maintaining framing action through the structure. This concept has been formally embodied in API RP2A (1993) where earthquakes are a necessary inelastic design consideration for US waters. The RP2A commentary relates to the proportioning of members (and joints) to provide adequate ductility. Although these guidelines were introduced to cover earthquake loading scenarios, they are also clearly related to considerations of the ultimate response and post ultimate behaviour of structures under extreme storm loading.
The multiplicity of load paths, may enhance the behaviour of the system such that the failure of a single member does not necessarily lead to catastrophic structural collapse. This is attributed to the redundancy of the system but it is demonstrated below that careful definition of the term is required.

In conventional deterministic structural engineering, redundancy is generally equated to the degree of indeterminacy, i.e. the number of unknown internal member forces in excess of the number of degrees of freedom of the system. However, this definition does not account for the existence of a weak link in an otherwise highly redundant system or the redistribution of redundancy throughout the system.

Lloyd and Clawson (1984) suggest that, in practice, each member should systematically be removed so that the consequences, in terms of the remaining capacity beyond which progressive collapse occurs, can be evaluated. In this way the concept of residual strength was developed. They present the hierarchy reproduced in Table 2.1 to demonstrate the gradation in redundancy that can be afforded by different members. Such an approach can be used as a basis for sizing members to give adequate redundancy.

<table>
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<th>Member Redundancy Level</th>
<th>Member Classification</th>
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<tr>
<td>0</td>
<td>A member whose failure leads to progressive collapse for dead weight load conditions.</td>
</tr>
<tr>
<td>1</td>
<td>A member whose failure leads to progressive collapse for dead plus some fraction of live weight load conditions.</td>
</tr>
<tr>
<td>2</td>
<td>A member whose failure leads to progressive collapse for a limited set of load conditions that include dead and live loads in combination with some fraction of the design environmental load.</td>
</tr>
<tr>
<td>3</td>
<td>A member whose failure leads to progressive collapse for a limited set of load conditions that include dead and live loads in combination with some multiple of the design environmental load.</td>
</tr>
<tr>
<td>4</td>
<td>A member whose failure has little effect on the design strength, but whose presence enhances the redundancy of nearby members, i.e. a normally lightly loaded member that provides an alternative load path when a nearby member fails.</td>
</tr>
<tr>
<td>5</td>
<td>A member whose failure has no bearing on the design, reserve or residual strength, i.e. a nonstructural member.</td>
</tr>
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In 1979 Marshall proposed two alternative measures of redundancy. For simple systems with a number of identical parallel load carrying elements (Npx), a redundancy factor (REF) was defined as:
REF = \frac{\text{damaged strength}}{\text{strength loss}} \quad \text{represented by} \quad \frac{N_{LP} - 1}{N_{LP} - (N_{LP} - 1)} = N_{LP} - 1 \quad \text{Eqn 2.5}

Values of REF less than unity therefore imply a high likelihood that initial failure will lead to collapse, whereas very high values relate to damage tolerant structures.

The alternative measure is the damaged strength ratio (DSR) given by:

\[ \text{DSR} = \frac{\text{damaged strength}}{\text{intact strength}} = \frac{N_{LP} - 1}{N_{LP}} \quad \text{Eqn 2.6} \]

For more complex structures where \( N_{LP} \) is not directly available, the effect of damage is established by comparing the results of structural analyses for the intact and damaged structures. Frangopol and Curley (1987) proposed the use of a strength redundant factor (SRF) defined as:

\[ \text{SRF} = \frac{\text{intact strength}}{\text{intact strength} - \text{damaged strength}} \quad \text{Eqn 2.7} \]

In later work by Stewart et al (1988) another definition of redundancy, again denoted RF, is defined as:

\[ \text{RF} = \frac{\text{ultimate structure resistance}}{\text{structure resistance at which first member fails}} \quad \text{Eqn 2.8} \]

In some ways this definition may be considered to be more akin to the foregoing definitions of reserve strength. Nevertheless, the notation indicates the strong correlation between redundancy and system reserve. Caution in determining first member failures is also required, however. It might be considered as the first occurrence of plasticity which needs to be defined in terms of complete section or extreme fibre conditions, or otherwise might be linked to buckling and a loss of a component capacity. Recent use of the term 'System Redundancy Factor' (SRF) (Banon, 1994), has referred to first major member failure, to avoid reference to early failure of a secondary component which plays no part in the overall system response. It should be noted that in some instances first component failure may not necessarily be related to a member, and tubular joints and foundations may need to be considered.

In the jacket study by Nordal et al (1988) probabilistic measures were introduced and additional consideration of these is given in Sections 2.2.10 and 2.2.12. Taking \( \beta_{\text{system}} \) and \( \beta_{1} \), respectively, as the safety index for the full system and for the union of first member failures (i.e. the combined probability of any member failing first), a redundancy measure:

\[ \frac{\beta_{\text{system}} - \beta_{1}}{\beta_{\text{system}}} \quad \text{Eqn 2.9} \]

is proposed. For a statically determinate system \( \beta_{\text{system}} = \beta_{1} \), and therefore redundancy is given as zero, whereas for a highly redundant system \( \beta_{\text{system}} \gg \beta_{1} \), such that this redundancy measure would approach unity.
Nordal et al also present a more direct measure of redundancy based on the conditional probability of system failure given any first member failure. This latter definition relates the failure probabilities for the system and union of first member failures, as for the safety index approach above. The authors also suggest that in some circumstances the conditional failure probability can be approximated by the ratio of probabilities associated with the most-likely-failure-path to the most-likely-to-fail-first-member which is easier to obtain than the combined probabilities.

The ISO code (ISO 2394) provides a qualitative definition of redundancy as the ability of a structure to find alternative load paths following failure of one or more non key elements.

The redundancy measures reviewed in this section aim at measuring system load path redundancy, which may enhance the behaviour of the system such that the failure of a single member does not lead to catastrophic structural collapse. System redundancy is a function of various factors including panel redundancy, plan framing redundancy and local joint redundancy. Panel redundancy, where there is a multiplicity of load paths in one panel, may not necessarily lead to system redundancy. Other types of redundancy include joint redundancy, where for example in overlapped K-joints the applied load is partially transferred from one brace to the other through their common weld. Another type of joint redundancy is the capacity of the joint to transmit loads after the failure of various members connected to it. This is a function of the configuration and total number of members connected to a joint. It should be recognised from the above discussion that designers and analysts must examine various factors to ensure an adequate degree of system redundancy.

On the basis of the various redundancy measures proposed to date it is difficult to draw generalised conclusions. It should be noted that many of these measures are load case dependent and a structure may exhibit very different redundancy properties for different loading directions. For example, in one direction the structure may be able to mobilise out of plane bracing to shed load whereas for an orthogonal direction this may not be the case. Further consideration of structural configurations and orientation will be given in the reviews that follow.

### 2.2.5 Residual strength and robustness considerations

The concept of residual strength is particularly important in assessing the capacity of a structure which has been damaged, be it due to accidental loading, fatigue, fracture, or extreme environmental loads. More recently, it is increasingly being used to assess the necessity for inspecting various members. An important factor which contributes to the residual strength of offshore jackets is the portal frame action. In some cases, it may be possible to achieve higher values of residual strength by choosing sections with higher plastic moment capacities.

Lloyd and Clawson (1984) define residual strength in terms of a Residual Resistance Factor (RIF) given by:

$$RIF = \frac{\text{ultimate strength of damaged structure}}{\text{ultimate strength of undamaged structure}}$$  \hspace{1cm} \text{Eqn 2.10}

It should be recognised that the value of the residual strength corresponds to a particular displacement, and that if the load is increased further a different value of residual strength may be achieved. Hence, when comparing different structural configurations a clear definition of 'collapse in the damaged state' should be used (eg. Structural load corresponding to twice the ultimate displacement).
Figures 2.1 and 2.2 illustrate the definition of reserve and residual strength for a structure subjected to increasing applied loads. The frame design load is the load at which the most highly utilised component reaches its maximum allowable stress (in WSD terminology) or at which the summation of loads multiplied by the relevant load factors reaches the strength multiplied by the resistance factors (in LRFD terms), again for the most highly utilised component. Therefore the frame design load may be expressed as the load at first component failure (Point C in Figure 2.1) divided by the safety factor. The ultimate capacity of the frame is defined as the maximum load that may be sustained by the jacket (Point A in Figure 2.1). The RSR ratio is defined as the ultimate capacity of the frame divided by the design load (A/D). Once the ultimate capacity of the frame is reached, load shedding occurs and the maximum load that may be sustained is reduced (Point B in Figure 2.1). Point B should not be confused with the residual strength of the jacket, which is illustrated in Figure 2.2. When the load is removed, the damaged jacket will return to a new 'deformed' position (Point F in Figure 2.2) rather than to its original undeformed configuration (Point E in Figure 2.2). Under a new loading condition, the damaged jacket will deform in a more flexible manner. The residual strength is defined as the ultimate capacity of the jacket, when subjected to loading from a damaged state (Point B' in Figure 2.2). Generally, Points B and B' have similar values. However, it should be recognised that loads are not necessarily shed from damaged components to give the same force distribution as if the loads were applied to the damaged structure from an unloaded case.

The frame design load defined in the above expressions refers to the most highly utilised component with a utilisation ratio of 1.0. However in design practice, component utilisation ratios are often limited to a value below 1.0 (for example, 0.6 for legs under still water conditions, 0.85 and 0.8 for legs and braces respectively under storm conditions. See Section 3.3.1). Therefore, the design load obtained using standard design practice should be correlated to the definition of the design load used in this section. This may be done by introducing a "correction factor" to be applied to the design loads such that the most highly utilised component will have a utilisation ratio of 1.0. In this manner, frames having different maximum component utilisation ratios may be compared from a common basis.

Nordal et al (1988) adopted an alternative probabilistic view of residual strength termed 'robustness'. The probability of system failure in the presence of damage compared with the intact structure is defined as the robustness factor. The lower the robustness factor, the less effect the component failure has had on the system.

The term robustness is also used by Stewart (1992 a and b) to compare the post-ultimate and ultimate system strengths as a measure of resilience or a structure's ability to dissipate energy through nonlinear hysteretic cycles.

The ISO code (ISO 13819) provides a qualitative definition of robustness as the ability of a structure to withstand accidental events or consequences of human error, without being damaged to an extent disproportionate to the original cause.

Frequently the residual strength is estimated by removing 'damaged' members (eg. Piermattei et al, 1990) or by introducing damaged member properties (eg. Martindale et al, 1989) and performing a new analysis. If acceptable data are available the latter approach is to be preferred as it will more accurately reflect the load distribution through the structure. In the first instance, the concern is that although the approach may be conservative from a local viewpoint, it may not lead to conservatism in predicting the overall nonlinear collapse behaviour of the structure. Furthermore, it may be necessary to consider the sequence of loading, component failure and redistribution, as removing members and repeating the analysis
from the unloaded state may not yield the same load distribution and hence post ultimate response.

2.2.6 Ductile versus brittle responses
In the context of overall structural performance, the terms 'ductile' and 'brittle' may be used to identify the characteristics of the response. If the global capacity is maintained or continues to increase despite a 'component' failure, the behaviour is said to be ductile. If rapid unloading occurs, i.e. a reduction in capacity) the response is described as brittle. At a component level ideal tensile behaviour is clearly ductile (provided the material is ductile), whereas the rapid load shedding associated with fracture is brittle. The more gradual unloading of tubular beam columns is an intermediate case where the reduced residual capacity is described as semi-brittle. Marshall and Bea (1976), for example, use the terms further in the descriptions of 'brittle-redundant' and 'ductile-redundant' structures and responses.

The term ductility is also used in another context for problems such as seismic loading or impact. Ductility is an important property for consideration of energy absorption. A ductility ratio is used to characterise plastic deformation capability and is the ratio of total available deformation to initial peak elastic deformation. Energy absorption capacity is equivalent to the area under the load-deformation curve for the structure and is also related to the post-peak capacity as discussed in Section 2.2.5.

It should be recognised that energy absorption capacity, as defined above as equal to the area under the load deflection curve, is dependent on the starting point (intact versus damaged structure) of the ultimate strength analysis (or a corresponding laboratory test). Figure 2.2 shows the load deflection curves for a structure loaded from both an intact state and a damaged state. Despite the fact that both structural configurations have identical residual strengths, the energy absorption capacity of the damaged structure may be significantly reduced.

2.2.7 Considerations of Structural Optimisation and Redundancy
Traditionally speaking, research activity on, and practical interest in, structural optimisation has been targeted towards finding the optimum design corresponding to the jacket structure of least weight. However, structural optimisation may significantly reduce the reserve strength and the degree of redundancy of the structure. For example, the deletion of members with low elastic utilisation ratios to save weight reduces the capacity of redistribution along alternative load paths. Indeed, under any one particular loading scenario, the jacket structure of least weight must be statically determinate. Often, linear analysis of structures, will result in negligible load in the horizontals between the panels. However, in the event of damage to diagonal braces, these horizontals would be essential for maintaining framing action through the structure. More recently, the possible adverse effects of structural optimisation have been recognised and, therefore, weight strength-redundancy optimisation schemes are now more commonly used.

Lloyd (1982) carried out a study on idealised three dimensional structures to determine the minimum structural weight corresponding to a particular value of residual strength. Linear elastic analyses showed that both the horizontal members in the vertical frames and the diagonal bracing in plan framing levels are redundant. However, in the event of a diagonal brace 'failure' in the vertical frames, the horizontal and plan bracing members provide important alternative load paths to distribute the forces efficiently down through the structure.

Feng and Moses (1986) proposed the use of reserve strength-dependent optimisation criteria for sizing components of a structure subjected to both extreme and accidental loading conditions. The required magnitudes of reserve and residual strength were considered to be
a function of structure-dependent parameters including the extreme and accidental loads acting on the structure, the frequency of inspections to identify damaged members and the time required for any essential repairs.

Along similar lines, Frangopol and Klisinski (1989) proposed the use of a weight-strength redundancy optimisation scheme and applied it to study the behaviour of an idealised three dimensional jacket. Brittle, ductile and ductile-brittle material behaviour were considered. By carrying out strength analyses on the optimised and original structure, they concluded that optimised structures show lower values of reserve, residual and redundancy margins for all types of material behaviour. In addition, the ultimate strength of the optimised structure was shown to be more sensitive to variations in component strength.

Probabilistic optimisation schemes aimed at balancing least weight, highest system reliability and highest system redundancy are also being developed and their use in offshore structure applications will be discussed in Sections 2.2.10 and 2.2.12.

2.2.8 Sources of Uncertainties and Sensitivities

Over the past few decades, a significant number of studies have been carried out to classify and examine the sources of uncertainties associated with the determination of the ultimate strength and post ultimate behaviour of structures. The sources of uncertainty may be classified in the following main categories:

- Variations in component strength including effects of effective lengths, imperfections, defects and damage
- Uncertainties in the loading conditions
- Uncertainties in the modelling of the foundation
- Uncertainties in the modelling of the joints.

The main contributing factor to variations in component strength is the uncertainty in the yield strength which has a coefficient of variation of around 10% (Frangopol and Curley, 1986). Loading uncertainties arise from the random nature of the sea, and are often treated as stochastic processes. However, the sensitivity of the ultimate strength of a jacket to the hydrodynamic loading distribution may be studied in a deterministic manner by varying the ratio of the still water to the environmental loading.

Uncertainties in the modelling of the foundation are due to the lack of sufficient data on the pile model and soil parameters. However, with the large number of experimental and analytical studies currently being carried out, it is expected that this uncertainty may be reduced. The uncertainty associated with the modelling of the joints has reduced significantly as a result of several joint industry projects (eg. BOMEL Document No. C63632060R).

In addition to the four types of uncertainties listed above, variations in component strengths and uncertainties associated with the sequence of component failure lead to the following two types of system effects:

- Probabilistic effects associated with the uncertainty in the system behaviour due to variations in component strength.
- Deterministic effects associated with the sensitivity of the redundancy in the system to variations in the component behaviour (eg sequence of component failure). A structure may exhibit very different redundancy properties for different loading directions. For example, in one direction the structure may be able to mobilise out-of-
plane bracing to shed load whereas for an orthogonal direction this may not be the case.

Section 2.2.9 addresses issues regarding deterministic sensitivity studies, while probabilistic performance measures are discussed in Section 2.2.10.

2.2.9 Performance measures based on deterministic sensitivity studies

The traditional measures of redundancy, discussed in Section 2.2.4, treat redundancy as being independent of structural response sensitivities. More recently new types of definitions, where the structure may have varying degrees of redundancy corresponding to the response sensitivity under consideration, have emerged. For example, a structure may have varying degrees of redundancy depending on the sequence of component failure. Frangopol and Curley (1987) interpret the general definition of redundancy as the absence of primary components whose failure would lead to catastrophic structural collapse. Based on the strength redundant factor defined by Equation 2.7, they identify the consequences of damage to various members. Components that significantly reduce the redundancy in the system are identified for additional quality assurance, inspection and maintenance. While the above procedure has been developed to determine the degree of redundancy in bridges, it is equally applicable to offshore structures.

Using a similar approach, Frangopol and Klisinski (1989) assessed the effect of different component damage on the system strength of an idealised three dimensional truss-system. Again, it was proposed that members whose absence significantly reduces the redundancy of the system should be identified as primary members requiring a more rigorous design, inspection and maintenance. While the above studies form a significant step forward in recognising the dependence of redundancy on the sequence of failure, they fall short of outlining a detailed methodology for carrying out sensitivity studies.

Pandey and Barai (1997) proposed a systematic approach for determining the redundancy of the structural system based on the sensitivity of its components. Response sensitivities of a structural component reflect changes in its response with respect to a damage parameter (e.g. the collapse of a member). The response sensitivity of a component may be expressed mathematically as the derivative of one of its response parameters (e.g. stress, strain or displacement) with respect to a damage parameter. In this manner, they derived expressions for response sensitivities based on the global finite element equations. The lower the value of an element sensitivity, the higher its level of redundancy.

It should be recognised that according to the definition above, the redundancy of a particular structure is not a fixed quantity, but rather depends on the damage parameter and associated response of the structure at a given stage. While the method discussed above provides a systematic approach for determining redundancy as a measure of structural sensitivities, it is yet to be applied to analyse practical offshore structural configurations. Until then, its efficiency and accuracy cannot be accurately assessed.

On the basis of the above sensitivity-based redundancy performance measures, the way forward seems to lie in carrying out a variety of analyses to assess the effects of various parameters on the ultimate and post-ultimate behaviour of structures. Values of residual and reserve strength based on a single ultimate strength analysis are dependent on the loading, sequence of component failure and damage parameters corresponding to the analysis under consideration and cannot be generalised to other conditions.
2.2.10 Probabilistic Performance Measures

The past two decades have witnessed a significant increase in the use of probability based design procedures for building structures and nuclear power plants. More recently, the offshore industry has also investigated the use of probabilistic concepts in the design process. Indeed, this is reflected by the development of international standards for the use of reliability analysis (ISO 2394). While there have been significant advances in this field, there are still issues which should be addressed by both practitioners and researchers. This will facilitate the emergence of a probabilistic methodology that is capable of responding to the needs of the industry. Paliou et al (1990) and Cornell and Edwards (1992) identified the following main outstanding issues:

- Development of methods for system reliability analysis of complex offshore structures including foundations.
- Development of methods for estimating the joint occurrence of environmental variables and loading conditions.
- Development of a general methodology for setting performance targets and acceptance criteria.
- Development of probabilistic performance measures.

The present review addresses recent advances regarding the development and use of probabilistic performance measures for the offshore industry.

Research activity on the development of probabilistic performance measures has been motivated by the fact that deterministic performance measures, such as redundancy when expressed in terms of the degree of indeterminacy, do not properly reflect the overall system strength, are independent of important structural parameters such as member material behaviour and do not account for uncertainties in the loading and material strength.

In Section 2.2.4, the redundant strength factor was used by several researchers to identify primary members, whose performance is key to the system performance. It was then proposed that these primary members should be inspected more frequently than non-critical members. This approach was extended by Frangopol and Curley (1987) into the probabilistic domain, by introducing the following probabilistic damage factor:

\[ DF = \frac{S_{\text{intact}} - S_{\text{damaged}}}{S_{\text{intact}}} \]  

Eqn 2.11

where

- \( S_{\text{intact}} \) is the mean intact strength of member \( i \)
- \( S_{\text{damaged}} \) is the mean damaged strength of member \( i \)

Next, a probabilistic performance measure is defined on an element level as:

\[ M_{ij} = S_i - Q_{ij} \]  

Eqn 2.12

where

- \( M_{ij} \) is the probabilistic performance measure of element \( i \) corresponding to a particular damage level in element \( j \)
- \( Q_{ij} \) is the random load in element \( i \) corresponding to a particular damage level in element \( j \)
Finally, for each member $i$, a safety index is defined as:

$$\beta_{ij} = \frac{M_{ij}}{\sigma(M_{ij})}$$  \hspace{1cm} \text{Eqn 2.13}$$

where

- $M_{ij}$ is the mean value of the performance measure of element $i$ corresponding to a particular damage level in element $j$.
- $\sigma(M_{ij})$ is the standard deviation of the performance measure of element $i$ corresponding to a particular damage level in element $j$.

The safety index of the damaged structural system, $\beta_{\text{system}}$, is obtained from the safety indices $\beta_{ij}$. This system safety index is bounded by both member and failure mode correlations. The system safety index of the intact structure may be determined in a similar way.

In this manner, Frangopol and Curley (1987) arrived at a probabilistic measure of redundancy, $\beta_R$, defined as:

$$\beta_R = \frac{\beta_{\text{intact}}}{\beta_{\text{intact}} - \beta_{\text{system}}}$$  \hspace{1cm} \text{Eqn 2.14}$$

where

- $\beta_{\text{intact}}$ is the safety index of the intact system.

Paliou et al. (1990) introduced a slightly different probabilistic measure of redundancy, expressed in terms of the probability, $P_r$, that the structure will survive the simultaneous failure of one or more of its members. This probability of survival takes into account uncertainties in loading conditions and variations in material and component strength.

Fu and Frangopol (1990) proposed the use of an optimisation framework which takes into account the minimum weight requirements, system reliability and redundancy. To this end, they introduced a probabilistic redundancy index, $RI$, defined as:

$$RI = \frac{P_{f(dmg)} - P_{f(sys)}}{P_{f(sys)}}$$  \hspace{1cm} \text{Eqn 2.15}$$

where

- $P_{f(sys)}$ is the probability of system collapse.
- $P_{f(dmg)}$ is the probability of a component failure.

The difference between the probability of component failure and the probability of system collapse corresponds to the probability of having a redundant system. By carrying out optimisation studies using the above definition, they concluded that when both system reliability and system redundancy are considered, a more rational design may be obtained.
Fu and Moses (1989) identified a gap in system reliability measures as applied to intermediate states between an intact structure and total collapse. They introduced a component-dependent probabilistic redundancy measure, the component redundancy index (CRI), defined as:

$$\text{CRI}_c = \frac{P_{f_{(00)}} - P_{f_{(0)}}}{P_{f_{(0)}}}$$

Eqn 2.16

where $P_{f_{(0)}}$ is the probability of system failure when component C is completely damaged.

The higher the value of CRI, the more critical component C is to the integrity of the system.

It can be seen from the above discussion that a variety of deterministic and probabilistic measures have appeared in the literature underlying the lack of universally accepted definitions.

The main advantage of reliability-based definitions of performance measures is that uncertainties in the loading, variations in component strength and variations in the sequence of failure are treated in a probabilistic manner. While probabilistic performance measures clearly provide more insights into the behaviour of the structure than sensitivity-independent deterministic measures, their comparative advantages with respect to sensitivity-based deterministic measures will be discussed in Section 2.2.12.

2.2.11 Sensitivity-based versus sensitivity-independent deterministic approaches

Both sensitivity-independent and sensitivity-based deterministic approaches are being adopted to investigate the collapse behaviour of jacket structures. The sensitivity-independent deterministic approach is to perform a static pushover analysis, using specific nonlinear software, to compare the peak and post-ultimate capacities of the structure with the design load. The member properties, geometry and loading are considered to have unique values.

In sensitivity-based deterministic approaches various parameters including the hydrodynamic loading distribution and direction, member material properties, joint flexibility, foundation flexibility, initial imperfections, bracing configuration and degree of damage to the structure are varied to determine their effect on the ultimate strength and post-ultimate behaviour of the structure.

In this manner, lower and upper bounds to the various performance measures may be obtained. Furthermore, members may be classified as critical or non-critical according to the effect of variations in their component strength on the structural system strength. Critical members can then be identified to receive more detailed quality assurance, inspection and maintenance schemes.

Clearly, the sensitivity-based deterministic approach provides more insights into the behaviour of the structure. The main question, therefore, that analysts and designers should address is whether to use a sensitivity-based deterministic approach or a reliability-based one. This issue is discussed in the next section.

2.2.12 Reliability-based versus deterministic sensitivity-based approaches

Both deterministic sensitivity-based and reliability-based approaches are being increasingly used to study the ultimate and post-ultimate behaviour of offshore structures. The
Characteristics of the sensitivity-based deterministic approach have been discussed in Sections 2.2.9 and 2.2.11.

In reliability based approaches member properties, geometry and loading are considered as variables with known or assumed statistical distributions. Simplified structural assessments are performed to identify 'important' sequences of component failures, i.e. sequences which have a high probability of occurrence. The results are generally presented in terms of either the probability of occurrence, P, or the safety index, \( \beta \). The two measures may be considered to be equivalent based on the relation:

\[
P = \Phi(\beta)
\]

Eqn 2.17

where

\( \Phi() \) is the cumulative normal distribution.

It should be recognised that both approaches referred to in this section attempt to determine upper and lower bounds to structural performance measures and to identify critical members. The main difference is in the methods which are adopted to represent the variations in structural and loading parameters. While the reliability based approach adopts statistical distributions, the analyst/designer must choose which properties to vary in the sensitivity-based deterministic approach. This high degree of user interaction required by the latter approach should not necessarily qualify as a disadvantage, particularly since adequate statistical and probabilistic methods for modelling the foundation and joint behaviour of practical offshore structures are not yet available.

Furthermore, it has been found that for intact structures, the failure mode and capacity established by a deterministic pushover analysis is usually an adequate representation of the structure for a probability analysis. The reason is attributed to the far higher uncertainty in environmental load than for resistance (Banon, 1994).

2.2.13 Pushover analysis and cyclic loading

Reserve strength is assessed in terms of a structure's ability to resist loads in excess of the design value. For a jacket structure it is typically evaluated by applying the maximum loading from the extreme event and performing a so-called 'pushover' analysis.

Although this static approach to collapse is now widely adopted, the relation between the models and the real situation needs to be reviewed. For an extreme storm the environmental loading is cyclic, imposed in an underlying dominant direction. The maximum wave is unlikely to be an isolated event, but will be a peak in a series of extreme loads. The possibility of cyclic degradation of components which have failed, or are near failure even though the overall structural resistance may remain adequate, therefore needs to be considered. Results published in 1993, based on studies of North Sea jackets, suggested that an extreme event static analysis may generally suffice to demonstrate a structure's resistance to the cyclic loading of a full storm. These results are reviewed below.

A set of four papers was presented by investigators drawn largely from SINTEF and Shell Research at the Offshore Mechanics and Arctic Engineering Conference in 1993 (Stewart et al, Stewart and Tromans, Eberg et al and Hellan et al) to establish whether strength estimates based on pushover analyses are suitable measures of system capacity.

The principal concern is whether cumulative damage due to cyclic loading will reduce system capacity below predictions from the single 'worst' event. In this regard cyclic loading may be
associated with the sequence of waves in a given storm (short-term effects) or with the re-
occurrence of storms over a longer time period (long-term effects). Stewart and Tromans
examined both short term and long term wave statistics and established that one extreme storm
may be considered to dominate the load history and this may be represented by factoring the
100 year ‘design storm’ to give a rare event with a notional 10,000 year return period. The
sequence of the diminishing waves within the design storm is shown to be modelled
conservatively by a ‘pseudo-storm’ comprising the most probable largest waves in the storm,
thereby ignoring short term real wave sequence effects. Were these taken into account, it is
shown that the second and subsequent waves in the storm would be smaller than those given
by the pseudo-storm.

The storm loading history for cyclic analysis parallels the single storm load applied in static
pushover analysis. Within the sequence, the 100 year wave loading is applied initially to
identify alternating plasticity at low storm intensities. The above extreme storm is then
factored (a factor of 1.5 corresponding to the 10,000 year event which would be representative
of Central North Sea long term statistics but not necessarily for other regions) with the
application of waves in descending order having been shown to be most damaging. Finally
the 100 year loading is re-applied as a stability check after the passage of the storm.

More recently, in Phase III of the BOMEL JIP Frames Project, a series of three-dimensional
tests were carried out to establish the effects of nonlinear joint/member behaviour and cyclic
loading on three dimensional frame behaviour and the reserve and residual strength of three
dimensional frames (ORF 103, 1994 and ORF 119, 1997). Results from the BOMEL JIP
project generally supported the above conclusions. However, depending on the strength and
configuration of the joints and the margin of reserve strength of the structure, cyclic loading
may significantly degrade the reserve strength obtained using an ultimate strength analysis.
This is particularly important in cases where the structure may experience significant plasticity
leading to high strain, low cycle fatigue cracking. It should be recognised that cyclic loading
generally results in a more severe degradation of joint strength than member strength
particularly for K joints. Therefore, it is important to account for joint stiffness and strength
when examining the effect of cyclic loading on the reserve strength of jackets. However in
most cases tested, degradation of joint strength during representative storm sequences such as
that outlined above did not undermine the system redundancy or residual strength.

2.2.14 Effects of bracing configurations in two-dimensional vertical frames
The ultimate strength of a structure depends on the nonlinear response of components within
a frame and the interaction between those components. It is generally accepted that within a
particular frame, X-bracing offers greater reserves than either the K-bracing or the single
diagonal bracing. However, the degree of reserve depends on the slenderness of the braces
and the redundancy throughout the structure. The API RP2A (WSD and LRFD) (1993)
guidelines address the issues of ductility requirements for structures in seismically active areas.
They provide various suggestions which, if not adhered to, may require the analyst/designer
to carry out a detailed ductility analysis. While these regulations were developed for
earthquake loading, they are also useful in assessing the redundancy of a structure and the
availability of alternative load paths in general. Indeed the objectives of the API guidelines on
ductility are stated as follows:

1. Provide sufficient system redundancy such that load redistribution and inelastic
deformation will occur before collapse.
2. Minimise abrupt changes in stiffness in the vertical configuration of the structure.
3. Provide for redistribution of the horizontal shear loads in the vertical frames as
   buckling occurs in the diagonal bracing.
4. Improve the post buckling behaviour of the diagonal braces.

The remainder of this section discusses these guidelines in view of the bracing configurations adopted in this study.

API RP2A-WSD Section C.4.3.2 recommends the following provisions to ensure adequate ductility:

1. K-bracing should not be used where the ability of a panel to transmit shear is lost if the compression brace buckles. Figure 2.3 shows two bracing configurations which satisfy (Figures 2.3a) and violate (Figure 2.3b) this requirement.

2. Diagonal bracing in the vertical frames is configured such that shear forces between horizontal frames are distributed approximately equally between compression and tension braces.

3. Diagonal bracing in the vertical frames are configured such that shear forces in vertical runs between legs are distributed approximately equally to both compression and tension braces.

4. Horizontal members are provided between all adjacent legs at horizontal framing levels, with the intention that these members will support the redistribution of loads resulting from the buckling of adjacent diagonal braces.

Figure 2.4 shows a four-legged frame which violates suggestions 2 to 4 above, while Figure 2.5 shows another four-legged frame which adheres to the API guidelines.

If failure occurs in a K-brace the load path through the panel is lost, the tension and compression braces are effectively in series and the response is brittle. In the X-braced case, if the compression brace buckles additional load can still be carried through the panel via the tension brace. So long as the stiffness of the tension brace exceeds the rate of unloading from the compression brace, the panel as a whole can take increasing global load. The members may be considered to act in parallel and the response is ductile.

If the braces are designed to the same codes the reserve strength of the K panel will equal the safety factor adopted, whereas for the X panel it may be greater due to the tension brace contribution. This is a result of frame action ignored in traditional elastic design.

The above discussion, together with the API guidelines, refer to the advantages of using a particular bracing configuration in two-dimensional vertical frames. It does not refer to the effects of portal action and multiplanar bracing configurations.

2.2.15 Effects of plan bracing configurations in three-dimensional jackets

Until recently very few studies have been carried out to determine the effects of multi-planar bracing configurations on the reserve strength of offshore jackets. In particular, the effects of various bracing configurations at plan framing levels have not been properly addressed.

Gebara et al (1998) carried out a study on two three dimensional platforms with different bracing schemes at plan framing levels. They concluded that in the presence of adequate bracing at plan framing levels, the absence of horizontal members in vertical frames does not significantly affect the response of the structure.

Figure 2.6 shows two vertical frames where Frame A has horizontal members at plan levels, while Frame B does not. By carrying out two dimensional pushover analyses on these frames, it is well established that Frame A may have a higher residual strength and a more robust post-
ultimate behaviour. Indeed the API guidelines refer to this point by recommending the use of horizontal members at plan framing levels.

However, an additional parameter (namely the bracing configuration at plan framing levels) should be considered to account for the three dimensional nature of the platforms. Figure 2.7 shows various bracing configurations at plan framing levels corresponding to a four-legged platform (where Points A, B, C and D lie at the same elevation but each on a different leg). Figure 2.7b satisfies API guidelines, while Figure 2.7a violates them. However, results from three dimensional pushover analyses carried out by Geba et al (1998) indicate that there is no significant difference between the two configurations. Figures 2.7c, d and e show three additional types of bracing configurations at plan framing level. It should be noted that the bracing in Figure 2.7c, where the diamond braces extend between the X-nodes in the corresponding vertical frames as per the Dunbar and Armada platforms, does not provide a high degree of redundancy in the system. Clearly, more studies should be carried out to assess the effects of multiplanar framing configurations and the recommended guidelines for ductility and redundancy should address these multiplanar effects.

### 2.3 SUMMARY AND CONCLUSIONS

In this section some key considerations in evaluating the reserve strength of jacket structures have been discussed. Their simple definition has been shown to be complicated by many factors including differences between alternative design processes (eg. LRFD v WSD) and structural configuration.

All the performance measures presented in the above sections are summarised in Table 2.2 below as a basis for selecting the measures to be used in this study.

<table>
<thead>
<tr>
<th>Performance Measure</th>
<th>Expression</th>
<th>Quantity Measured</th>
<th>Deterministic (D) / Probabilistic (P)</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>B (Eqn 2.1)</td>
<td>-√ (energy capacity / energy demand)</td>
<td>reserve strength</td>
<td>D</td>
<td>Blume, 1960</td>
</tr>
<tr>
<td>RSR (Eqn 2.2)</td>
<td>Ultimate platform resistance / design load</td>
<td>reserve strength</td>
<td>D</td>
<td>Titus &amp; Banon, 1988</td>
</tr>
<tr>
<td>REF (Eqn 2.5)</td>
<td>Damage strength / strength loss</td>
<td>redundancy</td>
<td>D</td>
<td>Marshall, 1979</td>
</tr>
<tr>
<td>DSR (Eqn 2.6)</td>
<td>Damage strength / intact strength</td>
<td>redundancy</td>
<td>D</td>
<td>Marshall, 1979</td>
</tr>
<tr>
<td>SRF (Eqn 2.7)</td>
<td>Intact strength / (intact strength - damaged strength)</td>
<td>redundancy</td>
<td>D</td>
<td>Frangopol &amp; Curley, 1987</td>
</tr>
<tr>
<td>RF (Eqn 2.8)</td>
<td>Ultimate structure resistance / structure resistance at first member failure</td>
<td>redundancy</td>
<td>D</td>
<td>Stewart et al, 1988</td>
</tr>
<tr>
<td>(Eqn 2.9)</td>
<td>(β_{sys} - β) / β_{sys}</td>
<td>redundancy</td>
<td>P</td>
<td>Nordal et al, 1988</td>
</tr>
<tr>
<td>Performance Measure</td>
<td>Expression</td>
<td>Quantity Measured</td>
<td>Deterministic (D) / Probabilistic (P)</td>
<td>Reference</td>
</tr>
<tr>
<td>---------------------</td>
<td>------------</td>
<td>------------------</td>
<td>--------------------------------------</td>
<td>-----------</td>
</tr>
<tr>
<td>RIF (Eqn 2.10)</td>
<td>Ultimate strength of damaged structure / ultimate strength of intact structure</td>
<td>residual strength</td>
<td>D</td>
<td>Lloyd and Clawson, 1984</td>
</tr>
<tr>
<td>RS</td>
<td>Residual strength of intact structure / ultimate strength of intact structure</td>
<td>residual strength</td>
<td>D</td>
<td>OTH 92 365</td>
</tr>
<tr>
<td>$\beta_x$ (Eqn 2.18)</td>
<td>$\beta_{\text{instruct}} / (\beta_{\text{instruct}} - \beta_{\text{assistance}})$</td>
<td>redundancy</td>
<td>P</td>
<td>Frangopol &amp; Curley, 1987</td>
</tr>
<tr>
<td>RI (Eqn 2.19)</td>
<td>$(P_{\beta_{\text{instruct}}} - P_{\beta_{\text{assistance}}}) / P_{\beta_{\text{assistance}}}$</td>
<td>redundancy</td>
<td>P</td>
<td>Fu &amp; Frangopol, 1990</td>
</tr>
<tr>
<td>CRI (Eqn 2.20)</td>
<td>$(P_{\beta_{\text{instruct}}} - P_{\beta_{\text{assistance}}}) / P_{\beta_{\text{assistance}}}$</td>
<td>redundancy</td>
<td>P</td>
<td>Fu &amp; Moses, 1989</td>
</tr>
<tr>
<td>$W_g$</td>
<td>N/A</td>
<td>weight of jacket</td>
<td>D</td>
<td>N/A</td>
</tr>
<tr>
<td>$n_\omega$</td>
<td>Total number of welds associated with bracing</td>
<td>fabrication effort/cost</td>
<td>D</td>
<td>N/A</td>
</tr>
<tr>
<td>B (Eqn 2.1)</td>
<td>Energy capacity / energy demand</td>
<td>energy absorption</td>
<td>D</td>
<td>Blume (1960)</td>
</tr>
<tr>
<td>DR</td>
<td>Total deformation / initial peak elastic deformation</td>
<td>ductility</td>
<td>D</td>
<td>N/A</td>
</tr>
<tr>
<td>R</td>
<td>Robustness</td>
<td>robustness</td>
<td>D</td>
<td>Stewart et al (1992a and b)</td>
</tr>
</tbody>
</table>

While performance measures related to ductility and energy absorption are referred to separately in the above table, it should to recognised that these two quantities are not independent.

When deciding on the choice of performance measures to be used in assessing the results of ultimate strength analyses, the major decisions facing the analyst include:

1. Most suitable quantity which should be measured, depending on the purpose of the ultimate strength analysis. For example, when carrying out pushover analyses to determine the effect of various bracing configurations on the available degree of redundancy in the system, performance measures which reflect the redundancy should be chosen while those reflecting reserve strength above are considered inadequate.

2. The need to carry out sensitivity studies, for example, when attempting to determine which members require a more detailed degree of inspection, the effect of various member damaged on the reserve strength and redundancy in the system should be examined.

3. The choice of deterministic-based or probabilistic-based sensitivity studies. This depends on the capabilities of available software and the purpose of the ultimate strength analyses.
In the context of this study, it is required to compare the pre-ultimate and post-ultimate performance of the various framing configurations. To this end, reserve strength, residual strength and redundancy-based performance measures will be used. To achieve a high degree of confidence in the results, both energy based and load based performance measures will be used.

The following performance measures were selected to be tested for a range of structural frames:

- $R_1 = \text{environmental load at ultimate} / \text{environmental load at first plastic hinge normalised}$
- $R_2 = \text{environmental load at ultimate} / \text{environmental load at first component failure (RF)}$
- $R_3 = \text{environmental load at twice the ultimate deflection} / \text{environmental load at ultimate (RS)}$
- $R_4 = \text{energy at environmental ultimate load} / \text{energy at first member failure}$
- $R_5 = \text{energy at twice the ultimate deflection} / \text{energy at environmental ultimate load}$
- $R_6 = \text{weight of jacket} (W_c) - (\text{kN})$
- $R_7 = \text{fabrication cost} (\text{\pi} \alpha)$
- $R_8 = \text{reserve strength ratio (RSR)}$

The first, second and fourth performance measures are an indication of the available reserve strength up to the point where the ultimate load of the jacket has been reached. They are hereafter referred to as pre-ultimate performance measures as they measure the degree of redundancy in the system up to ultimate. The third and fifth performance measures reflect the degree of redundancy in the system after the ultimate load has been reached. They are hereafter referred to as post-ultimate redundancy measures. To compare frames from a common basis, the residual strength and the energy are measured at twice the ultimate deflection.

The sixth and seventh performance measures ($R_6$ and $R_7$) which will be used in this study provide an indication to the cost and level of difficulty of the fabrication process, which increases as the weight and the number of welds is increased.

The load factor at ultimate will also be used and it may be considered as a measure of RSR, however it should be recognised that load factors derived from computer pushover analyses are a ratio of the total environment load divided by the design load.

In addition, as discussed in Section 2.2.5, the load factor is applied to design loads which do not lead to utilisation ratios of 1.0 in the case of the most highly utilised component. Therefore, a correction will be applied to the design loads such that the most highly utilised component will have a utilisation ratio of 1.0. In this manner the RSR ($R_8$) is defined as the environmental load at collapse $\div$ (design load x correction factor).

The energy at twice the ultimate deflection (and the energy at ultimate) are calculated as the area under the load deflection curve. Load incrementation schemes provide the most realistic modelling of environmental loading; however, they become numerically unstable in the post-ultimate range and often displacement-control methods are used instead. This inconsistency relates to both post ultimate performance measures ($R_5$ and $R_7$) and it should be recognised that it is a limitation which is common to all numerical-based studies.
It should be recognised that pre-ultimate and post-ultimate performance measures refer to separate structural characteristics. Both energy-based and load-based pre-ultimate and post-ultimate performance measures are determined from the analyses reported herein, validated and used to assess the degree of redundancy in different framing configurations.

In this study, sensitivity studies were carried out to examine the effects of initial imperfections, strain hardening, joint flexibility and distribution of hydrodynamic loading on the response. Deterministic based rather than probabilistic based performance measures were used. This is mainly because it was concluded that for intact structures, the failure mode and capacity established by a deterministic pushover analysis is usually an adequate representation of the structure (Banon, 1994). The reason is attributed to the far higher uncertainty in environmental load than for resistance.

The importance of variations in both wave length and height with return period is recognised; however, these effects are considered outside the scope of this project.
Figure 2.1
Definitions of reserve and residual strength - Loading from an intact state

Figure 2.2
Definitions of reserve and residual strength - Loading from a damaged state
Figure 2.3
Permissible and non-permissible K-bracing configurations - two legged frame

Figure 2.4
Non-permissible bracing - four legged frame
Figure 2.5
An example of a bracing configuration which satisfies API guidelines

Figure 2.6
Vertical bracing configurations
Figure 2.7
Plan bracing configurations
3. SELECTION AND DESIGN OF FRAMING CONFIGURATIONS

3.1 SELECTION OF FRAMES

3.1.1 Introduction
An extensive survey of North Sea structures has revealed that there are many and wide variations in structural configuration, bracing patterns, number of members, structural weight in relation to water depth, topsides pay load, etc. There is also a wide variation in overall structural utilisation depending on the 'optimisation' of the design. A structural configuration classification method was adopted and jackets were separated into types according to various factors, including:

- Water depth
- Number of legs
- Bracing configuration (X, X with horizontals, K, inverted K, single diagonal)
- Number of bays.

A study of 140 UK Sector platforms has identified 80 different framing configurations (transverse and horizontal) within 3, 4, 6 and 8 legged structures with between three and eight horizontal framing levels.

There are several small variations on a small number of themes and therefore a representative range of behaviour could be captured by the selection of the 20 frame (configurations) shown in Figures 3.1 to 3.6.

As will be discussed in Sections 4, 5 and 6, the frames which are not symmetric (9 out of 20) will be analysed under two different loading directions which brings the total number of base cases to 29.

When selecting the frame geometries, special emphasis was placed on choosing a wide range of X and K bracing configurations. While the bracing configurations are representative of existing structures in the UK North Sea sector, only some of these configurations satisfy the API ductility requirements.

The main features of the premise on which the structures were designed is discussed below.

Parameters concerned with the geometry and modelling of the jackets are presented in the remainder of this section. These include the relationship between water depth, number and spacing of bays, leg batter and frame dimensions.

3.1.2 Bay Spacing, Number of Bays and Water Depth
The number of vertical bays in a jacket structure is generally related to the water depth. For this study, jacket frames with two to five vertical bays were examined and the relationship with water depth shown in Table 3.1 was adopted. This relationship is representative of steel jacket structures in the UK sector of the North Sea.
Table 3.1
Number of vertical bays and water depth

<table>
<thead>
<tr>
<th>Number of vertical bays</th>
<th>Water depth (metres)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>25.0</td>
</tr>
<tr>
<td>3</td>
<td>40.0</td>
</tr>
<tr>
<td>3</td>
<td>80.0</td>
</tr>
<tr>
<td>4</td>
<td>100.0</td>
</tr>
<tr>
<td>5</td>
<td>140.0</td>
</tr>
</tbody>
</table>

In general, bay spacing is controlled by the ability of conductors, risers and caissons to span the bay height. It is also generally observed from past designs that bay spacings decrease near the water surface, where wave and current action result in a greater magnitude of lateral loading than at locations nearer the seabed. Examination of older generation jacket configurations indicates smaller bay spacings compared to more slender recent designs. Table 3.2 presents those bay spacings used in this study.

Table 3.2
Bay spacing

<table>
<thead>
<tr>
<th>Number of bays</th>
<th>Water depth (metres)</th>
<th>Elevation of jacket top plan above LAT (metres)</th>
<th>Bay spacing from top plan level (metres)</th>
<th>Distance from bottom plan level to seabed (metres)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>25</td>
<td>+4.6</td>
<td>11.6, 17.0</td>
<td>1.0</td>
</tr>
<tr>
<td>3</td>
<td>40</td>
<td>+4.6</td>
<td>11.6, 15.0, 17.0</td>
<td>1.0</td>
</tr>
<tr>
<td>3</td>
<td>80</td>
<td>+10.0</td>
<td>25.0, 30.0, 33.0</td>
<td>2.0</td>
</tr>
<tr>
<td>4</td>
<td>100</td>
<td>+10.0</td>
<td>25.0, 25.0, 28.0, 30.0</td>
<td>2.0</td>
</tr>
<tr>
<td>5</td>
<td>140</td>
<td>+10.0</td>
<td>25.0, 28.0, 28.0, 32.0, 35.0</td>
<td>2.0</td>
</tr>
</tbody>
</table>

3.1.3 Number of Legs and Frame Batter
The majority of UK sector North Sea jackets comprise 4, 6 or 8 legs. Table 3.3 presents the typical conditions that were investigated as part of the two-dimensional computer analyses for this study. Batters for the legs of 1:20 and 1:10 were used for the longitudinal and transverse frames respectively. For the purpose of the analyses, the plane of the 2-D frames was assumed to be vertical.

Table 3.3
Number of legs

<table>
<thead>
<tr>
<th>Number of bays</th>
<th>Water depth (metres)</th>
<th>Number of legs</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Transverse frame</td>
</tr>
<tr>
<td>----------------</td>
<td>----------------------</td>
<td>-----------------</td>
</tr>
<tr>
<td>2</td>
<td>25</td>
<td>✔</td>
</tr>
<tr>
<td>3</td>
<td>40</td>
<td>✔</td>
</tr>
<tr>
<td>3</td>
<td>80</td>
<td>✔</td>
</tr>
<tr>
<td>4</td>
<td>100</td>
<td>✔</td>
</tr>
<tr>
<td>5</td>
<td>140</td>
<td>✔</td>
</tr>
</tbody>
</table>
3.1.4 Jacket Top Dimensions
Table 3.4 shows the dimensions at the stabbing-in point at the jacket top adopted in this study; the stabbing-in point was assumed to be 1.0m above the top framing level. The dimensions have been based on average geometries of several UK sector North Sea platforms.

Table 3.4
Jacket top dimensions

<table>
<thead>
<tr>
<th>Total number of legs</th>
<th>Water depth (metres)</th>
<th>Longitudinal dimensions at top of jacket, l (metres)</th>
<th>Transverse dimensions at top of jacket, b (metres)</th>
<th>( l/b )</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>≤ 40</td>
<td>14.0</td>
<td>14.0</td>
<td>1.0</td>
</tr>
<tr>
<td></td>
<td>&gt; 40</td>
<td>24.0</td>
<td>24.0</td>
<td>1.0</td>
</tr>
<tr>
<td>8</td>
<td>≤ 40</td>
<td>31.0</td>
<td>14.0</td>
<td>2.2</td>
</tr>
<tr>
<td></td>
<td>&gt; 40</td>
<td>52.0</td>
<td>24.0</td>
<td>2.2</td>
</tr>
</tbody>
</table>

3.1.5 Topsides Modelling
A simple 2-D truss was modelled to account for the structural interaction of the topsides.

The bottom flange of the lower beam (Cellar Deck) was set at a distance of 3.0m above the crest of the 100-year wave. The upper beam was assumed to be 6.5m above the lower beam. The upper and lower beams were taken as 1.0m deep plate girders, and the bracing as 0.5m diameter tubulars.

3.1.6 Support Conditions
Skirt piles are required for large structures carrying significant topsides weight and exposed to large environmental forces. Where topsides and environmental loads are smaller and where soil conditions permit, leg piles may be used provided they have sufficient strength to resist the applied loads.

The main objective of this study is to examine the structural performance of framing configurations and therefore extensive soil and support modelling is considered unnecessary. However, to ensure that the analyses were representative of overall stiffness behaviour, support conditions noted in Table 3.5 were adopted.

Table 3.5
Support conditions

<table>
<thead>
<tr>
<th>Number of bays</th>
<th>Water depth (metres)</th>
<th>Support condition</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>25</td>
<td>Leg piles through corner legs welded at top elevation and shimmmed at all other plan levels. Piles extend six diameters below mudline where they are fully fixed.</td>
</tr>
<tr>
<td>3</td>
<td>40</td>
<td>Single, equivalent stiffness pile member at each corner leg extending from half height of bottom bay to six diameters below mudline where full fixity is provided.</td>
</tr>
<tr>
<td>5</td>
<td>100</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>140</td>
<td></td>
</tr>
</tbody>
</table>
3.2 SELECTION OF LOADING

3.2.1 Introduction
The two dimensional frames were analysed elastically using the SACS programme for the following conditions:

- still water condition
- 100-year storm with the wave positioned for maximum base shear.

The distribution of the loading may affect the collapse mode of the jacket. Therefore, it is important to select realistic topside wave and current loading conditions. The remainder of this section outlines the methodology adopted in selecting the above parameters.

3.2.2 Topsides Weight
Topsides weights shown in Table 3.6 were used for this study. The loading was considered to be distributed equally at each leg location and was applied as point loads.

<table>
<thead>
<tr>
<th>Water depth (metres)</th>
<th>Total number of legs</th>
<th>Total topsides weight (tonnes)</th>
</tr>
</thead>
<tbody>
<tr>
<td>25.0</td>
<td>4</td>
<td>1200</td>
</tr>
<tr>
<td>40.0</td>
<td>8</td>
<td>2400</td>
</tr>
<tr>
<td>80.0</td>
<td>8</td>
<td>16000</td>
</tr>
<tr>
<td>100.0</td>
<td>8</td>
<td>16000</td>
</tr>
<tr>
<td>140.0</td>
<td>8</td>
<td>16000</td>
</tr>
</tbody>
</table>

Member self-weight and buoyancy for non-flooded members were evaluated automatically using the SACS program.

3.2.3 Environmental Loading
Hydrodynamic Loading
Hydrodynamic loading will be evaluated for storm waves and currents acting in the plane of the two-dimensional framings using the parameters in the following sections. For asymmetric frames the hydrodynamic loading was applied in both directions.

The wave and current loading were evaluated using the SACS program. For frame design the hydrodynamic loading on the primary modelled members was applied as distributed loads. Equivalent nodal loads and moments were derived for use with SAFJAC for the design wave crest positioned relative to the structure to produce maximum base shear.

Wind loading is typically only 5% to 10% of the lateral 100-year hydrodynamic load and was therefore neglected.

Wave Height and Period
Typical 100-year return storm wave heights and periods for the selected water depths used in the analyses are shown in Table 3.7.
For the purpose of the analysis the water level will be assumed to be at LAT.

<table>
<thead>
<tr>
<th>Water depth, d (metres)</th>
<th>H max (metres)</th>
<th>T max (seconds)</th>
<th>H/gT^2</th>
<th>d/gT^2</th>
<th>Wave Theory</th>
</tr>
</thead>
<tbody>
<tr>
<td>25</td>
<td>14.1</td>
<td>12.0</td>
<td>0.0100</td>
<td>0.0177</td>
<td>Stream function V</td>
</tr>
<tr>
<td>40</td>
<td>15.2</td>
<td>14.0</td>
<td>0.0079</td>
<td>0.0208</td>
<td>Stream function V</td>
</tr>
<tr>
<td>80</td>
<td>25.0</td>
<td>16.0</td>
<td>0.0100</td>
<td>0.0319</td>
<td>Stream function V</td>
</tr>
<tr>
<td>100</td>
<td>28.0</td>
<td>15.0</td>
<td>0.0127</td>
<td>0.0453</td>
<td>Stokes V</td>
</tr>
<tr>
<td>140</td>
<td>30.0</td>
<td>15.0</td>
<td>0.0136</td>
<td>0.0634</td>
<td>Stokes V</td>
</tr>
</tbody>
</table>

Current Profile
The omnidirectional currents used, consisting of tide and storm surge acting with the 100-year wave, are given in Table 3.8; these were applied in-line with the wave.

<table>
<thead>
<tr>
<th>Water depth, d (metres)</th>
<th>Surface current (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>25</td>
<td>0.75</td>
</tr>
<tr>
<td>40</td>
<td>0.8</td>
</tr>
<tr>
<td>80</td>
<td>0.9</td>
</tr>
<tr>
<td>100</td>
<td>1.0</td>
</tr>
<tr>
<td>140</td>
<td>1.2</td>
</tr>
</tbody>
</table>

The current profile was assumed to be constant through the top 75% of the water column, reducing linearly to zero at the mudline. The current was stretched nonlinearly from mean water level to the crest, and the Doppler effect of the current on the wave period was taken into account, as recommended by API.

Drag and Inertia Coefficients
For the purpose of the analysis, constant drag and inertia coefficients for use with Morison's equation were taken as:

\[ C_d = 0.65 \quad C_m = 1.6 \quad \text{smooth members (assumed above LAT)} \]
\[ C_d = 1.05 \quad C_m = 1.2 \quad \text{rough members (assumed below LAT)} \]

Current blockage and directional spreading factors were taken as 1.0 and marine growth thickness was ignored.

Appurtenances and Out-of-Plane Members
In order to account for the hydrodynamic loading on the conductors, risers, and other appurtenances, six 0.6m diameter conductors were considered. They were laterally supported at the Cellar Deck and at each plan framing level, and were assumed to be fully fixed at six diameters below the mudline. The hydrodynamic loading was distributed to the adjacent legs at each horizontal framing position.
The hydrodynamic load acting on the out-of-plane members was evaluated approximately by considering 3 or 4 dummy elements at each horizontal framing position. The size and length of the dummy elements was assumed commensurate with the members in the modelled frame. The same modelling of out-of-plane members was assumed for all frames at each water depth.

3.3 FRAME DESIGN

In order to understand the effects of framing arrangements, component strength and failure characteristics it is desirable to compare alternatives from an equivalent basis. In order to ensure realistic equivalence, the frames and component details were configured using a common design approach as outlined below.

3.3.1 Characteristics of the Common Design Approach

Before comparing the redundancy and capacity for load redistribution in structural framing it is important to establish baseline designs on a consistent basis.

The objective of the study is to consider the behaviour of realistic structures, and not to examine impractical designs or designs that have been fully optimised for one load condition only. In practice, many members are sized for a number of reasons, including temporary fabrication and installation conditions, the support of appurtenances, or for anticipated future expansion. These reasons cannot be explicitly considered in a study of this type, and thus a simple sizing approach was established.

Member Sizing

For each water depth a leg diameter (inner diameter for through-leg piles) was chosen for the frames by trial and error. Wherever possible a consistent leg diameter was used for all frames in the same water depth. (For the shallower water structures with through-leg piles this dimension will also determine the pile diameter.)

Brace diameters were selected based on the following guidelines:

- Minimum slenderness ratio \((KL/r) = 30\) (for primary members),
- Maximum slenderness ratio \((KL/r) = 120\),
- \(\beta\) ratios \((d/D) = 0.9\) for bracing K joints, and
- \(\beta\) ratios \((d/D) = 1.0\) for X joints.

API recommendations for effective lengths were followed; typically effective length factors are \(K = 1.0\) for legs, \(K = 0.9\) for X-braces, and \(K = 0.8\) for K-braces.

Member wall thicknesses were chosen by trial-and-error such that they lead to API RP2A utilisation ratios for design which are less than or equal to the following target values:

- Under still water / self weight conditions:
  Target utilisation for legs = 0.6

- Under storm conditions (wave and current loading) with one-third over-stress allowed:
  Target utilisation for legs = 0.85
  Target utilisation for braces = 0.8
In deriving the member utilisations for member design the effects of hydrostatic pressure were considered.

Brace, leg and can wall thicknesses satisfy the following criteria:

- Minimum diameter-to-thickness ratio 20,
- Maximum diameter-to-thickness ratio 65, and
- Minimum thickness will be 12mm.

Symmetry was considered when choosing member and can sizes. In addition, excessive variation in member sizes was avoided.

**Joint Can Sizing**

Joint stresses were checked to API RP2A-WSD. Where the maximum utilisation ratio in a joint was greater than 0.8 a joint can was designed. The can length was designed to meet API requirements, and the thickness was adjusted to give a maximum utilisation of 0.8. Fatigue analyses were not undertaken.

**3.3.2 Implementation of the Design Approach**

The frames were designed using the SACS program. For non-symmetric frames, load conditions from two different directions were considered.

For each case, an iterative procedure was followed to ensure all the conditions described above were satisfied.

The case of the 25m deep, transverse Frame C (see Figure 3.7, where the out-of-plane members used for calculating the corresponding hydrodynamic loading are also shown) is discussed below.

Figure 3.7 shows the computer model plot with the node numbers. Tables 3.9 and 3.10 show a sample of the member and joint utilisation ratios respectively. Load cases 5 and 6, referred to in the above tables denote the still water and the storm loading respectively.

Throughout the design process, similar tables were generated by SACS for each frame. The utilisation ratios or 'unity checks' were examined to ensure the design premise was satisfied. If any utilisation ratio violated the conditions set in the design premise the corresponding member was redesigned.

Examination of Tables 3.9 and 3.10 reveals that all the conditions outlined in Sections 3.3.1 have been met.

Elastic analyses are not reported in detail as they only form the basis for establishing representative jacket framing arrangements for which the ultimate response characteristics are investigated.
Table 3.9

Member utilisation ratios

<table>
<thead>
<tr>
<th>Member</th>
<th>Utilisation Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Member 1</td>
<td>0.345</td>
</tr>
<tr>
<td>Member 2</td>
<td>0.456</td>
</tr>
<tr>
<td>Member 3</td>
<td>0.567</td>
</tr>
<tr>
<td>Member 4</td>
<td>0.678</td>
</tr>
<tr>
<td>Member 5</td>
<td>0.789</td>
</tr>
</tbody>
</table>

... (Continued)
Table 3.10
Joint utilisation ratios
Figure 3.1
25m 2-bay frames

Figure 3.2
40m and 80m transverse 3-bay frames

Figure 3.3
100m transverse 4-bay frames
Figure 3.4
40m and 80m longitudinal 3-bay frames

Figure 3.5
100m longitudinal 4-bay frame

Figure 3.6
140m longitudinal 5-bay frames
Figure 3.7
Computer plot for 25m transverse
4. STRUCTURAL ANALYSIS APPROACH

4.1 STRUCTURAL MODEL

Nonlinear pushover analyses of the jackets were carried out using SAFJAC. The types of element and the method of modelling joint flexibility and strength are described in the following sections.

4.1.1 Types of element

SAFJAC has five types of element for three-dimensional modelling of offshore platforms. They allow accurate modelling of:

- Elastic large deflection beam-column behaviour (Type 33)
- Plastic hinges (Type 34)
- Distributed plasticity (Types 31 and 32)
- Joint flexibility and soil behaviour (Type 41).

The main features of the elements are:

Type 31: Cubic elastic-plastic element for modelling material nonlinearity. The formulation is based on numerical integration and monitors stresses over the cross-section of the member at each Gauss integration point along the member. The member behaves according to the material stress-strain relationship and the formulation accounts for nonlinear geometric effects. The element is less accurate than Types 32-34 and therefore more elements are required to represent nonlinear geometric behaviour.

Type 32: Quartic elastic beam-column element with automatic subdivision on detection of plasticity. One element is usually sufficient to model a complete member. On subdivision, Type 31 elements are inserted into the plastic zones while the elastic zones are kept as Type 32. With this type of element, different values of strain hardening may be adopted.

Type 33: Quartic elastic beam-column element. One element is generally sufficient to represent the beam-column effect and the elastic large displacement response of a whole member.

Type 34: Quartic elastic beam-column element that allows plastic hinges to form at one or both ends of the element. If requested, the element will automatically subdivide into two elements of Type 34 when a plastic hinge is detected within the element length. For elements that yield in tension, it may be necessary to prevent subdivision. This type of formulations ignores the effects of strain hardening.

Type 41: Nonlinear spring element for modelling local joint flexibility and strength as described in Section 6.3.

4.1.2 Modelling of joint and soil nonlinear behaviour

Local joint flexibility and strength may be modelled in SAFJAC by using joint 'spring' elements which allow a piecewise linear load-deformation curve to be specified for each of the six local degrees of freedom \((u, v, w, \theta_x, \theta_y, \theta_z)\) corresponding to the six local load...
components ($F_x, F_y, F_z, M_x, M_y, M_z$).

Three types of load-deformation curve are available as shown in Figure 4.1. They are:

Type 1 - linear curve where only one parameter, $K_1$, is required to define the slope, i.e.: stiffness $K = P/\Delta$.

Type 2 - three-part piecewise linear curve for which five parameters, $K_1, \Delta_1, K_2, \Delta_2, K_3$, are required to define a trilinear response.

Type 3 - five-part piecewise linear curve for which nine parameters, $K_1, \Delta_1, K_2, \Delta_2, K_3, \Delta_3, K_4, \Delta_4, K_5$, are required to define the behaviour of a joint.

For the Type 1 and Type 2 springs, the parameters are assumed the same for both compressive and tensile forces. With the Type 5 springs, different five-part curves can be specified for tensile and compressive behaviour, thus allowing different ultimate strengths in tension and compression to be modelled. The Type 5 curve allows better modelling of the end bearing and T-Z stiffnesses of soil which may differ according to whether the pile is in tension or compression.

4.1.3 Computer models

The focus of the study is structural framing and therefore to limit the failure mode to elements in the jacket, the soil and piles were modelled using high strength material behaviour.

As will be discussed in Section 5, two different element formulations were used for all the base analysis cases. Both the plastic hinge formulation (Type 34 elements) and the distributed plasticity formulation (Types 31 and 32) were used to model the jacket.

The piles inside the legs, from the top of the jacket to the mudline, were modelled using quartic plastic hinge elements (Type 34) without element subdivision. Spring elements (Type 41) were introduced between the piles and legs to restrain the lateral movement of the pile to that of the leg. This is achieved by enforcing normal compatibility of displacements in the jacket and the piles at the leg-pile interface, while allowing for tangential discontinuity. In this manner a 'sliding' condition is achieved at the leg-pile interface.

4.2 MATERIAL PROPERTIES

The following material properties were used for all cases:

- Steel density = 7.85 tonnes/m³
- Young's modulus = 210 kN/mm²
- Poisson's ratio = 0.3
- $F_y$:
  - $t < 16$ mm: $F_y = 355$ N/mm²
  - $16$ mm $< t \leq 40$ mm: $F_y = 345$ N/mm²
  - $40$ mm $< t \leq 63$ mm: $F_y = 340$ N/mm²
  - $63$ mm $< t \leq 100$ mm: $F_y = 325$ N/mm²
Figure 4.1
Nonlinear springs material behaviour
5. RESULTS OF BASE ANALYSES

5.1 INTRODUCTION

This section reports the results of fifty-eight nonlinear ultimate strength analyses. The first twenty-nine analyses were carried out with the plastic hinge formulation, assuming rigid joint behaviour, no initial imperfections and no strain hardening. Results based on the assumption of rigid joint behaviour provide an upper bound to the response. However, this assumption is representative of modern structures, where the joints are designed to be stronger than the members and the braces.

A second set of twenty-nine analyses was carried out with a distributed plasticity formulation, assuming rigid joint behaviour, no initial imperfections and no strain hardening.

The plastic hinge formulation is more efficient in terms of computer time. In addition, it is easier to define first component failure. A compression member fails if it develops three plastic hinges, while a tensile member fails if it develops two plastic hinges along its length. The distributed plasticity formulation is, numerically, more stable when tracing the post ultimate behaviour of the jacket.

Nonlinear analyses produce large amounts of data which should be carefully examined to provide a sufficient degree of confidence in the results and to interpret the behaviour of the structure. The variables which are usually monitored are discussed below:

- Plastic hinge development and location: A global failure mechanism may be established by examining the location of the plastic hinges. Member failure may also be established by examining the location of plastic hinges along its length.
- P-Y and T-Z utilisation ratios: when the behaviour of the jacket is governed by the strength of the foundation, it is important to determine whether the P-Y or T-Z soil stiffness is governing the response.
- Pile plastic interaction ratios: if the foundation is governing the response of the jacket system, foundation failure often leads to the development of plastic hinges in the piles. A pile plastic interaction ratio of 1.0 indicates that the pile cross-section at a particular location has reached its plastic capacity.
- Load displacement curve: this is used to provide insights into the level of nonlinearity in the response and the ductility of the jacket.
- Deformed shape of jacket: this is used to check against unreasonable modes of deformation and to assist in determining the causes of the jacket deformation.
- Variation of axial forces in the legs: this is used in conjunction with the next item to provide insights into alternative load paths and the level of stress in the legs.
- Variation of spring forces in flexible joints: this is used to provide insights into load shedding (joint forces start decreasing), alternative load paths and the level of stress in the joints.

In addition, the performance measures presented at the end of Section 2.3 will be determined for each case. The pre-ultimate performance measures will be calculated using results based on the plastic hinge approach, while the post-ultimate performance measures will be determined using results based on the distributed plasticity solution.
In all cases, the analysis was continued for each case until one or more of the following conditions was reached:

- Solution numerical instability.
- Residual strength less than design load.
- Maximum displacement reaches twice the deflection at the ultimate load.

5.2 25M DEEP FRAMES

Five 25 metre, two bay frames were considered. Frame A, shown in Figure 5.1, has K-joints with bracing on one side only. Therefore, if the compression brace buckles, the load path through the bracing is lost. Indeed, this type of K-joint is classified as 'non-permissible' in the API guidelines on ductility requirements for offshore structures in seismically active areas. Pushover analyses were carried out on all five frames. The load deflection curve for Frame A is shown in Figure 5.2, where it can be seen that the load factor reaches a maximum value of 3.03. The first plastic hinge develops at the load factor of 2.91 before the ultimate load is reached, while the first member failure occurs in the bottom compression brace at a load factor of 2.95, after the ultimate load is reached. Figure 5.2 shows the rapid fall off in load (Point A) once the compression brace in the bottom bay buckles (see also Figure 5.3). Very little energy is used by the structure in deforming from its ultimate strength position to the first plateau. A second rapid fall off in load takes place (Point B) after the compression brace in the top bay buckles (Figure 5.4). The legs acquire the load shed by the compression brace and enable the overall structure residual strength to remain constant (Point C) as the deflections proceed due to portal frame action in the stiff squat legs.

Figure 5.3 shows the variation of the axial force in the bottom bay braces and adjacent horizontal members, with the horizontal displacement at the top right hand corner of the frame (Node 4). The compression brace in the bottom bay (Member 17-15) starts shedding load with the development of the first plastic hinge along its length. Consequently, the tension brace in the bottom bay (Member 17-14) sheds load to satisfy the vertical equilibrium condition at Joint 17. Figure 5.3 also shows the axial force variation in the horizontal Members 17-12 and 17-13. Figure 5.4 shows the axial force variation in the top bay braces, and adjacent horizontal members, with the horizontal displacement at the top right hand corner of the frame (Node 4). The axial forces in the compression brace (Member 16-13) and the tension brace (Member 16-12) increase after the bottom braces start shedding load. The axial load in these members is increased until the compression brace develops a plastic hinge and starts shedding load. It should be recognised that the tension braces shed load to satisfy the vertical equilibrium conditions at Joints 16 and 17. This is due to the framing configuration, and does not correspond to these braces reaching their tensile strength. The residual load in the damaged members is controlled by the plastic hinge capacity and P-Δ effects.

Frame B, which is unsymmetric as can be seen in Figure 5.5, has one diagonal brace in each bay. When the environmental loading is applied in the positive X direction, the bottom brace is in tension, while the upper brace is in compression and vice versa. If the compression brace buckles, the load path through the bay is lost, and due to the near vertical orientation of the jacket legs, the remaining tension brace will undergo load shedding to satisfy equilibrium at the common joint (Joint 13). If, however, the tension brace yields the load in the brace will remain constant and therefore the load path through the bracing will not be lost.
Figure 5.6 shows the load deflection curves for Frame B corresponding to environmental loading in both the positive and negative horizontal (x) directions. The positive case is considered first (25m-tranbp), where it can be seen that the load factor reaches a maximum value of 3.79. The first plastic hinge develops at a load factor of 3.28, while the first component to fail is the tensile brace in the bottom bay (member 13-14) at a load factor of 3.42. Figure 5.6 shows a decrease in stiffness after the tension diagonal reaches its yield capacity (Point A) (Figure 5.7). Rapid fall off in load occurs after the compression brace buckles (Point B). A larger amount of energy (in comparison with Frame A discussed earlier) is used by the structure in deforming from its ultimate strength to the first plateau. The intermediate horizontal member acquired the load shed by the compression brace and enabled the overall structure resistance to remain constant as the deflections proceeded.

The first increment in load (between Points A and B) is due to portal frame action in the bottom bay. The second increment in load (between Points C and D) is due to portal frame action of the whole structure.

Figure 5.7 shows the variation of the axial forces in the top and bottom braces, and in the adjacent horizontal members, with the horizontal displacement at the top right hand corner of the frame. It can be seen that the load in the compression brace (member 10-13) keeps on increasing after the tensile brace has yielded. Also, the axial force in the tensile brace remains constant after yielding. After the compression brace buckles and starts shedding load, the compressive axial force in the intermediate horizontal member is increased to resist a larger proportion of the load.

The case of the negative environmental loading (25m-tranbn) is considered next. Figure 5.6 shows the load deflection curve for Frame B corresponding to negative horizontal environmental loading, where it can be seen that the load reaches a maximum value of 3.37. The first plastic hinge develops at a load factor of 3.36 at joint number 13 in Member 13-14, while the first component to fail is the compression brace in the bottom bay (Member 13-14) at a load factor of 3.27 after the ultimate load is reached. Figure 5.6 shows the rapid fall off in load (Point A) once the first plastic hinge develops on the compression brace in the bottom bay buckles (Figure 5.8). The first subsequent increase in load (between Points E and F) correspond to the diagonal tensile brace and the horizontal member acquiring the load shed by the compression brace. The second increment in loads (Point F onwards) corresponds to portal frame action of the structure.

Figure 5.8 shows the variation of the axial forces in the top and bottom braces, and in the adjacent horizontal members, with the horizontal displacement at the top right hand corner of the frame. The compression brace in the bottom bay (Member 13-14) starts shedding load with the development of the first plastic hinge along its length. However, unlike the case of Frame A, the tensile brace continues to carry an increasing proportion of the load after the compression brace has buckled. This is possible because of the geometry of the legs and braces connected to Joint 13, which allows equilibrium to be satisfied after member 14-13 has buckled.

Frame C, which is shown in Figure 5.9, is an X braced frame. Unlike the frame in Figure 5.1, the compression and tension braces in this configuration act in parallel and therefore if one of the compression braces fail, the remaining braces should be able to transmit the load through the bay. Frame D, which is shown in Figure 5.10, is similar to Frame C, except that the horizontal member at intermediate plan framing level has been removed.
The load deflection curve for Frame C is shown in Figure 5.11, where it can be seen that the load reaches an initial maximum value of 2.97. The first plastic hinge develops at a load factor of 2.91, while the first component to fail is the bottom tensile brace in the bottom bay (Member 17-14) at a load factor of 2.96 before the ultimate load is achieved. Figure 5.11 also shows a rapid fall off in load (Point A) once the lower compression brace in the bottom bay buckles. The intermediate horizontal member and the compression and tension braces in the upper bay acquire the load shed by the compression brace in the lower bay and enable the structure to carry an additional load increment (between Points B and C). The plateau reached at Point C corresponds to the failure of the compression brace. The horizontal members acquire the load shed by the compression brace in the upper bay and enable the overall structure residual strength to remain constant as the deflections proceed.

Figure 5.12 shows the variation of the axial force in the bottom bay braces, and adjacent horizontal members, with the horizontal displacement at the top right hand corner of the frame. It should be recognised that the ultimate value is reached due to the formation of a plastic hinge in the compressive Brace 17-15. After the compressive brace starts shedding load, an alternative load path is mobilised through the intermediate horizontal component (Member 12-13). Also, the braces in the top bay carry a larger proportion of the load as the compression braces in the bottom bay buckle (Figure 5.13).

It can be seen that the X-bracing configuration with horizontal members at plan framing levels allows the frame to carry additional loading after the ultimate load is achieved.

Figure 5.11 also shows the load deflection curve for Frame D, where it can be seen that the load reaches a maximum value of 2.97. The first plastic hinge and the first component failure occurs in the bottom tensile brace in the bottom bay (Member 17-14) at a load factor of 2.96. The load deflection curve of Frame D follows the same path of that corresponding to Frame C up to a point well beyond the initial peak strength of the jacket (Point D). As seen in the case of Frame C, the failure of the compression brace in the lower bay causes the first rapid drop in load (Point A) and the increase in load after Point B corresponds to the lower horizontal member and the braces in the upper bay acquiring a higher proportion of the load. However, the absence of an intermediate horizontal member leads to an earlier failure of the compression brace in the upper bay (Point D). The subsequent increment in load (between Points E and F) is resisted by the horizontal members acquiring a higher proportion of the load. Beyond Point F, the legs acquire a greater proportion of the load and enable the overall structure residual strength to remain constant as the deflections proceeded due to portal frame action in the legs.

Figures 5.14 and 5.15 show the variation of the axial forces in the braces, and adjacent horizontal members, in the bottom and top bays respectively. While the bottom tensile brace is the first component to fail, it should be recognised that the maximum load is reached when a plastic hinge develops along the bottom compressive brace in the bottom bay (Members 17-15). Figure 5.15 shows that the bottom horizontal component (Member 14-15) is mobilised as an alternative compression load path after the bottom brace buckles. Despite the absence of an intermediate horizontal component, it seems that the X-bracing configuration leads to a system which does not suffer a large decrease in its load carrying capacity after the ultimate load is reached. However, examination of Figure 5.11 shows that intermediate horizontal components lead to a higher post-ultimate response.

Frame E, which is unsymmetrical as can be seen in Figure 5.16, has K-bracing in the bottom bay similar to that used in Frame A while the top bay has a single diagonal brace.
Figure 5.17 shows the load deflection curves for Frame E corresponding to environmental loading in both the positive and negative horizontal directions. The negative case is considered first (25m-tran-e) where it can be seen that the load reaches a maximum value of 3.16. The first plastic hinge develops at a load factor of 3.04, while the first component to fail is the compression brace in the bottom bay (Member 17-14) at a load factor of 3.01 after the ultimate load has been reached. Figure 5.17 shows a rapid drop in the load (Point A) after the compression brace is the bottom bay buckles. The first increment in the load (between Points B and C) is resisted by the horizontal members and the tensile brace in the upper bay. After the tensile brace in the upper bay yields (point C), the legs acquire a higher proportion of the load and enable the residual strength of the structure to increase as the deflection proceeds due to portal frame action in the legs.

Figure 5.18 shows the variation in the axial forces in the top and bottom braces, and adjacent horizontal members, with the horizontal displacement at the top right hand corner of the frame (Node 4). Again it can be seen that the compression brace in the bottom bay starts shedding load with the development of the first plastic hinge along its length. Consequently, the tension brace in the bottom bay (Member 17-15) sheds load to satisfy the vertical equilibrium condition at Joint 17. The horizontal component of the intermediate framing level (Member 17-13) develops a plastic hinge and begins shedding its load. After the compression brace buckles, the axial force in the tension brace in the upper bay (Member 10-13) increases to resist the applied loading.

The case for the positive loading (25m-tran-e-dp) is considered next. The load deflection curve is shown in Figure 5.17, where it can be seen that the maximum load reached is 3.1. The first plastic hinge develops at a load factor of 3, while the first component to fail is the compression brace in the bottom bay (Member 17-15). Examination of Figure 5.17 shows both responses are very close up to a point well beyond the maximum load has been reached. For the case of the negative loading the brace in the upper bay is in tension and therefore when it fails by yielding the axial force in the brace remains constant. However, for the case of positive loading, the upper brace is in compression and, it's buckling has a more severe effect on the global response of the frame.

The above discussion showed how different framing configurations affects the availability of alternative load paths in the system. The variation of the axial forces in the braces and intermediate horizontal components was presented and discussed in detail. In the remainder of this section, and the next four sections, various performance measures will be determined and used to rank framing configurations. To assess the degree of redundancy and reserve strength in the system, the following performance measures were determined:

- Environmental load factor at first plastic hinge formation.
- Environmental load factor at first member collapse.
- Environmental load factor at ultimate (reserve strength)
- Environmental load factor corresponding to twice the ultimate deflection.
- Energy corresponding to the ultimate deflection.
- Energy corresponding to first member failure.
- Energy corresponding to twice the ultimate deflection.

The following issues should be clarified regarding the load at ultimate and the reserve strength ratio ($R_u$):
1. The ultimate load is defined as the maximum load that may be sustained by the jacket.

2. The reserve strength ratio is usually defined as the ultimate load divided by the design load.

3. The design load, referred to in Point 2 above, is the load at which the most highly utilised component reaches its maximum allowable stress (in WSD terminology). Therefore the design load may be expressed as the load to cause first component failure divided by the safety factor. This definition of the design load is based on a component utilisation ratio of 1.0.

4. In the design premise, adopted in this study to reflect current design practice, utilisation ratios are limited to a value below 1.0. The allowable utilisation ratio usually depends on the loading condition (operating versus storm conditions) and the location of the component (bracing versus legs).

5. The results obtained from a pushover analysis are usually in the form of a load factor versus displacement. The load factor is applied to the environmental design loads, as defined in Point 4 above.

6. The measure of the reserve strength which will be adopted in this study is the ultimate environmental load divided by the environmental design load defined in Point 3 above.

7. A correction factor is used to obtain the environmental design load corresponding to a utilisation ratio of 1.0, given the environmental design load defined in Point 4 above.

In addition, the following issues should be addressed regarding the determination of the number of additional joint welds associated with the bracing:

1. Joint welds associated with intermediate horizontal members will be considered in the case of transverse frames but not longitudinal frames. This is mainly due to the fact that all the longitudinal frames considered in this study have intermediate horizontal members.

2. For deep water (80m, 100m and 140m) jackets, joint welds will be determined assuming the nodes are manufactured separately.

3. Joint welds associated with changes in the chord cross-sections at joint cans will not be considered.

To compare the various frames from a common basis, the following non-dimensional measures are introduced:

- $R_1 = \text{environmental load at ultimate / environmental load at first plastic hinge.}$
- $R_2 = \text{environmental load at ultimate / environmental load at first component failure (RF)}$
- $R_3 = \text{environmental load at twice the ultimate deflection / environmental load at ultimate (RS)}$
- $R_4 = \text{energy at environmental ultimate load / energy at first member failure}$
- $R_5 = \text{energy at twice the ultimate deflection / energy at environmental ultimate load.}$
- $R_6 = \text{weight of jacket (WJ) (kN)}$
- $R_7 = \text{fabrication cost (noa)}$
- $R_8 = \text{reserve strength ratio (RSR)}$
The first, second and fourth non-dimensional variables listed above are a measure of the pre-ultimate redundancy and ductility in the system, while the third and fifth non-dimensional variables are a measure of the post-ultimate ductility, redundancy and residual strength in the system.

Tables 5.1 and 5.2 present the actual and normalised performance measures for the 25 metre frames respectively. The second to fifth columns in Table 5.1 refer to non-dimensional values of the load factor, while the units of the last three columns correspond to the area under the load deflection curve.

### Table 5.1

**Performance measures for 25 metre frames**

<table>
<thead>
<tr>
<th>Frame / load</th>
<th>Load at first plastic hinge</th>
<th>Load at first member failure</th>
<th>Ultimate load</th>
<th>Load at twice ultimate deflection</th>
<th>Energy at first member failure</th>
<th>Energy at ultimate</th>
<th>Energy at twice the ultimate deflection</th>
</tr>
</thead>
<tbody>
<tr>
<td>Frame A</td>
<td>2.91</td>
<td>2.95 (C)</td>
<td>3.03</td>
<td>1.44</td>
<td>0.491</td>
<td>0.492</td>
<td>0.951</td>
</tr>
<tr>
<td>Frame B +ve</td>
<td>-</td>
<td>3.42 (T)</td>
<td>3.79</td>
<td>3.15</td>
<td>0.512</td>
<td>0.595</td>
<td>1.594</td>
</tr>
<tr>
<td>Frame B -ve</td>
<td>-</td>
<td>3.27 (C)</td>
<td>3.37</td>
<td>2.63</td>
<td>0.329</td>
<td>0.530</td>
<td>1.340</td>
</tr>
<tr>
<td>Frame C</td>
<td>-</td>
<td>2.96 (T)</td>
<td>3.34</td>
<td>2.97</td>
<td>0.404</td>
<td>0.405</td>
<td>1.215</td>
</tr>
<tr>
<td>Frame D</td>
<td>-</td>
<td>2.96 (T)</td>
<td>2.97</td>
<td>2.70</td>
<td>0.404</td>
<td>0.405</td>
<td>1.134</td>
</tr>
<tr>
<td>Frame E +ve</td>
<td>-</td>
<td>2.91 (C)</td>
<td>3.16</td>
<td>1.44</td>
<td>0.495</td>
<td>0.496</td>
<td>0.914</td>
</tr>
<tr>
<td>Frame E -ve</td>
<td>3.13</td>
<td>3.01 (C)</td>
<td>3.15</td>
<td>2.46</td>
<td>0.519</td>
<td>0.520</td>
<td>1.205</td>
</tr>
</tbody>
</table>

### Table 5.2

**Non-dimensional performance measures and ranking for the 25 metre frames**

<table>
<thead>
<tr>
<th>Frame / load</th>
<th>R₁</th>
<th>R₂</th>
<th>R₃</th>
<th>R₄</th>
<th>R₅</th>
<th>R₆</th>
<th>R₇</th>
<th>R₈</th>
</tr>
</thead>
<tbody>
<tr>
<td>Frame A</td>
<td>1.041</td>
<td>0.974</td>
<td>0.475</td>
<td>1.931</td>
<td>732</td>
<td>10</td>
<td>2.44</td>
<td></td>
</tr>
<tr>
<td>Frame B +ve</td>
<td>1.155</td>
<td>1.108</td>
<td>0.831</td>
<td>2.679</td>
<td>781</td>
<td>6</td>
<td>2.67</td>
<td></td>
</tr>
<tr>
<td>Frame B -ve</td>
<td>1.003</td>
<td>0.970</td>
<td>0.780</td>
<td>2.528</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Frame C</td>
<td>1.021</td>
<td>1.003</td>
<td>1.000</td>
<td>3.000</td>
<td>790</td>
<td>14</td>
<td>2.73</td>
<td></td>
</tr>
<tr>
<td>Frame D</td>
<td>1.021</td>
<td>1.003</td>
<td>0.780</td>
<td>2.800</td>
<td>801</td>
<td>12</td>
<td>2.48</td>
<td></td>
</tr>
<tr>
<td>Frame E +ve</td>
<td>1.040</td>
<td>0.921</td>
<td>0.456</td>
<td>1.842</td>
<td>734</td>
<td>8</td>
<td>2.48</td>
<td></td>
</tr>
<tr>
<td>Frame E -ve</td>
<td>1.006</td>
<td>0.956</td>
<td>0.781</td>
<td>2.318</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

R₁ = environmental load at ultimate / environmental load at first plastic hinge, R₂ = environmental load at ultimate / environmental load at first component failure (RF), R₃ = environmental load at twice the ultimate deflection / environmental load at ultimate (RS), R₄ = energy at environmental load at first member failure, R₅ = energy at twice the ultimate deflection / energy at environmental ultimate load, R₆ = weight of jacket (WJ) (kN), R₇ = fabrication cost (no), R₈ = reserve strength ratio (RSR).
It can be seen from the above tables that there is a very small margin of pre-ultimate redundancy. This may be due to the fact that the 25 metre jackets are two-legged two bay frames and therefore a component failure or yielding will significantly affect the ultimate load. However, very close differences were observed in the case of the post ultimate redundancy. The X-braced frames were found to have the highest post ultimate redundancy followed by the diagonal and finally the K-braced frames. When ranking unsymmetric frames, the loading direction which leads to the 'weakest' frame response is used.

Table 5.2 shows that all the frames have very close reserve strength ratios (Rv in Table 5.2). However, the frames have significantly different margins of post-ultimate redundancy and residual strength (Rr and Rs). The cost and the ease of fabrication, reflected by the performance measures Rs and Rr, are also shown in Table 5.2. It can be seen that Frame C which has the highest degree of redundancy has the highest number of brace-associated welds at joints. However, Frame C does not have the highest weight.

5.3 40M DEEP FRAMES

Five 40 metre, two and four legged frames were considered. Frame A, which is shown in Figure 5.19, is a two legged X-braced frame with horizontal members at plan framing. The compression and tension braces will act in parallel to resist the load. Furthermore, the intermediate horizontal members provide an alternative load path in case of brace failure.

The load deflection curve for transverse Frame A is shown in Figure 5.20, where it can be seen that the load factor reaches a maximum value of 3.20. The first plastic hinge develops at a load factor of 3.05 at Joint Number 16 in Member (16-20), while first member failure occurs simultaneously in the bottom compression brace (Member 17-20) and the bottom tension brace (Member 16-20) in the bottom bay at a load factor of 3.20. The X bracing configuration and the presence of horizontal members of plan framing levels allow the frame to carry a large proportion of its ultimate load after the first component failure. Figure 5.20 shows the rapid fall off in load (Point A) once the bottom compression and tension braces in the lower bay fail. The first increment in load (between Points B and C) is due to portal frame action in the lower bay. The intermediate horizontal members and the braces in the upper bays acquire the load shed by the braces in the lower bay, which enable the structure strength to remain constant as the deflections proceed (between Points C and D). The second rapid drop in load (between Points D and E) takes place after the lower intermediate horizontal member (Member (14-15)) buckles. The legs acquire the load shed by the lower intermediate horizontal member and enable the overall structure residual strength to increase (Point E onwards) due to portal frame action of the jacket.

Frame B, which is shown in Figure 5.21, is similar to Frame A, except that the horizontal members at intermediate plan framing levels have been removed. The load deflection curve for transverse Frame B is shown in Figure 5.22, where it can be seen that the load reaches a maximum value of 3.14. The first plastic hinge develops at a load factor of 3.11 at Joint Number 16 in Member (16-20), while the first member failure occurs simultaneously in the bottom compression brace (Member 17-20) and the bottom tension brace (Member 16-20) in the bottom bay at a load factor of 3.18 after the ultimate load has been reached. By comparing Figures 5.20 and 5.22, it can be seen that the absence of horizontal members leads to a larger drop in the post ultimate strength of the structure.

Figure 5.22 shows a rapid fall off in load (Point A) after the bottom compression and tension braces in the lower bay fail. Very little energy is used by the structure in deforming from its
ultimate strength position to the first plateau (i.e. between Points A and B). A second fall off in load takes place (Point C) after the lower compression brace in the intermediate bay fails. Again very little energy is used by the structure in deforming to a new plateau (i.e. between Points C and D). The remaining braces in the intermediate bay and those in the upper bay acquire the load shed by the bracing in the lower bay and enable the structure strength to increase at a small rate (stiffness) as the deflections proceed (between Points D and E). A third fall off in load takes place after the upper compression in the intermediate bay fails (Point E). The braces in the upper bay and the legs acquire the load shed by bracing in the intermediate bay, which enables the overall structure residual strength to remain constant (Point F onwards) as the deflections proceed due to portal frame action in the legs.

Tables 5.3 and 5.4 present the dimensional and non-dimensional performance measures for the 40 metre transverse frames respectively.

Again, it can be seen that there is a small margin of pre-ultimate redundancy available in the system. This may be due to the fact that the 40 metre transverse jackets are two legged frames and therefore load redistribution after a component failure leads to a drop in the maximum carrying capacity of the jacket. It should be recognised that pre-ultimate and post-ultimate redundancy measures reflect two separate phenomenon of structural behaviour. Indeed, examination of Table 5.4 reveals that Frame A has a very low margin of pre-ultimate redundancy and a high degree of post-ultimate redundancy. As expected, the presence of horizontal components at intermediate framing levels leads to a higher degree of redundancy in the system.

### Table 5.3
**Performance measures for 40 metre transverse frames**

<table>
<thead>
<tr>
<th>Frame</th>
<th>load at first plastic hinge</th>
<th>Load at first member failure</th>
<th>Ultimate load</th>
<th>Load at twice ultimate deflection</th>
<th>Energy at first member failure</th>
<th>Energy at ultimate failure</th>
<th>Energy at twice the ultimate deflection</th>
</tr>
</thead>
<tbody>
<tr>
<td>Transverse A</td>
<td>3.05</td>
<td>3.20</td>
<td>3.20</td>
<td>2.62</td>
<td>0.777</td>
<td>0.798</td>
<td>2.065</td>
</tr>
<tr>
<td>Transverse B</td>
<td>3.11</td>
<td>3.18</td>
<td>3.14</td>
<td>1.55</td>
<td>0.793</td>
<td>0.793</td>
<td>1.509</td>
</tr>
</tbody>
</table>

### Table 5.4
**Non-dimensional performance measures and ranking for the 40 metre transverse frames**

<table>
<thead>
<tr>
<th>Frame</th>
<th>R₁</th>
<th>R₂</th>
<th>R₃</th>
<th>R₄</th>
<th>R₅</th>
<th>R₆</th>
<th>R₇</th>
<th>R₈</th>
<th>Ranking of pre-ultimate redundancy</th>
<th>Ranking of post-ultimate redundancy</th>
</tr>
</thead>
<tbody>
<tr>
<td>Transverse A</td>
<td>1.049</td>
<td>1.000</td>
<td>0.819</td>
<td>1.027</td>
<td>2.658</td>
<td>915</td>
<td>22</td>
<td>2.67</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Transverse B</td>
<td>1.010</td>
<td>0.987</td>
<td>0.494</td>
<td>1.000</td>
<td>1.903</td>
<td>928</td>
<td>18</td>
<td>2.64</td>
<td>2</td>
<td>2</td>
</tr>
</tbody>
</table>

R₁ = environmental load at ultimate / environmental load at first plastic hinge, R₂ = environmental load at ultimate / environmental load at first component failure (RF), R₃ = environmental load at twice the ultimate deflection / environmental load at ultimate (RS), R₄ = energy at environmental ultimate load / energy at first member failure, R₅ = energy at twice the ultimate deflection / energy at environmental ultimate load, R₆ = weight of jacket (W₆) (kN), R₇ = fabrication cost (t), R₈ = reserve strength ratio (RSR).
Table 5.4 shows that all the frame have very close RSRs (R_d). However, the frame have very
different margins of post ultimate redundancy and energy absorption capacity (R_1 and R_2). The
ease of fabrication, and associated cost, reflected in the performance measure R_1 is highest for
Frame A which has the highest degree of post-ultimate redundancy and energy absorption
capacity. However, this is offset by the lower weights of Frame A in comparison to Frame B.

Figure 5.23 shows the unsymmetric four-legged longitudinal Frame A with K and diagonal
bracing. The configuration of the K-bracing is similar to that in Section 5.1 (Frame A), where
a buckling failure of a compression brace led to load shedding in the tension brace in order to
satisfy vertical equilibrium at the K-joints. The diagonal bracing in the middle vertical frame
is similar to that used in Frame B of Section 5.1, where a buckling failure in one of the
compression braces does not lead to load shedding in the adjacent tension brace. The diagonal
bracing configuration in the middle vertical frame is distributed between tension and
compression braces. Therefore, while the K-bracing configuration does not meet the API
guidelines, the diagonal bracing arrangement satisfies these recommendations.

The load deflection curves for longitudinal Frame A corresponding to environmental loading
in the positive and negative horizontal (x) directions are shown in Figures 5.24 and 5.25
respectively. The positive case (40m-longtap) is considered first, where it can be seen that the
load factor reaches a maximum value of 3.17. The first plastic hinge develops at a load factor
of 3.16 at Joint Number 25 in Member 25-30, while the first component to fail is the bottom
compression brace (Member 25-30) in the bottom bay of the middle panel at a load factor of
3.17.

Figure 5.24 shows the rapid fall off in load (between Points A and B) once the compression
brace in the bottom bay of the middle panel (Member 25-30) buckles. Very little energy is
used in deforming between Points A and B. The tension brace in the intermediate bay of the
middle panel acquires the load shed by Member 25-30 and yields. The K-bracing in the left
panel acquires the load shed by the braces in the middle panel and enables the overall structure
residual strength to increase (between Points B-C). A second fall off in load (Point C) takes
place after the compression brace in the bottom bay of the left panel fails (Member 24-54).
The K-bracing in the bottom bay of the right panel acquires the load shed by Member (24-54)
and enables the overall structure residual strength to increase (between Points D and E). A
third fall off in load (Point E) occurs after the compression and tension braces in the bottom
bay of the right panel fail (Member 26-57) and Member (57-27) respectively. The bottom
horizontal members acquire the load shed by the braces in the bottom right bay and enable the
structure strength to increase (between Points F and G). A fourth fall off in load (between
Points G and H) occurs after the bottom horizontal members begin to develop plasticity
(Member (54-29) and Member (57-31). The legs and the braces in the upper bays acquire the
additional load and enable the overall structure residual strength to increase (beyond Point H)
as the deflections proceed.

The case for the negative environmental loading is considered next. The load deflection curve
is shown in Figure 5.25, where it can be seen that the maximum load factor reached is 3.19.
The first plastic hinge develops at a load factor of 3.16 at Joint Number 25 in Member 25-30,
while the first component to fail is the tension brace (Member 25-30) in the bottom bay of the
middle panel at a load factor of 3.18.

Figure 5.25 shows the rapid drop off in load (Point A) after the tension brace in the bottom bay
of the middle panel (Member 25-30) and the compression brace in the middle bay of the
middle panel (Member 25-22) fail. The tensile brace in the lower bay of the right panel
Member (26-57), together with the compression brace in the bottom bay of the left panel
(Member 54-25)) also fail before the Point A is reached.
The K braces in the middle row acquire the load shed by the collapsed members and enable the overall structure residual strength to increase (between Points B and C). A second fall off in load occurs (Point C) after the compression and tension K bracing in the left bay of the middle row fail (Member (53-20) and Member (53-21)). The intermediate horizontal members and the K bracing in the middle bay of the right panel acquire the load shed by the braces in the middle bay of the left panel and enable the overall structure residual strength to increase (between Points D and E). A third fall off in load (Point E) occurs when the tension member in the middle bay of the right panel yields (Member 22-56). The remaining horizontal members and the bracing in the upper row acquire the load shed by the bracing in the middle row and allow the overall structure residual strength to remain constant (between Points F and G). Failure of several horizontal members lying on the second-from-bottom plan level results in a fourth fall off in load (between Points G and H). The legs acquire the load shed by the bracing and the horizontal members and enable the overall structure residual strength to remain constant as the deflections proceed due to portal frame action in the legs.

Figure 5.26 shows longitudinal Frame B, which has X and diagonal bracing. The bracing configuration in this frame satisfies all API ductility requirements.

The load deflection curve for Frame B is shown in Figure 5.27, where it can be seen that the load factor reaches a maximum value of 3.78. The first plastic hinge develops at a load factor of 3.69 in the middle of Member (30-71), while the first component to fail is the bottom compression brace, member 30-71) in the bottom bay of the middle panel, at a load factor of 3.72. It can be seen that an X-bracing configuration allows the frame to carry a large proportion of its ultimate load well beyond first component failure. Indeed, this is similar to the conclusions reached for the 25 metre X-braced frames (Frames C and D in Section 5.1) and 40 metre transverse frames discussed in this section. Figure 5.27 shows a gradual fall off (Point A) in load after various compression and tension braces in the lowest row fail. The remaining braces in the upper two rows acquire the load shed by the braces in the lowest row and enable the overall structure residual strength to remain constant as the deflections proceed due to portal frame action in the legs. To illustrate the effects of multiple panels on the availability of alternative load paths in the system, the variation of the member forces in the braces is discussed next.

Figure 5.28 shows the variation in the axial forces in the bottom bay of the left panel, where it can be seen that the tensile braces (members 29-71 and 26-71) continue to carry load after the compression braces buckle. A similar type of behaviour can be seen in Figures 5.29 and 5.30 which show the axial force variation in the braces in the bottom bays of the middle and right panels respectively. Figure 5.31 shows the axial force variation in the braces in the middle bay of the middle panel, where it can be seen that load redistribution starts taking place after the braces in the bottom bays reach their capacity. Finally, Figure 5.32 shows the axial force variation in the middle and top bays of the left and right panels, where it can be seen that after the compression brace in the middle bay of the left panel buckles (Member 20-25) load redistribution takes place and the compression brace in the top bay of the left panel resists an increasing proportion of the load. It can be seen from the above discussion that the main criteria for judging the availability of alternative load paths is independent of the number of legs, and panels, in a frame. One of the important conditions is that the buckling of a compression brace should not force any tensile braces in the same bay to shed load in, order to satisfy equilibrium conditions of a joint, before they reach their ultimate capacity. This criteria may always be used to judge the availability of alternative load paths, regardless of the number of bays and legs in a jacket.
Figure 5.33 shows longitudinal Frame C, which has an unsymmetric diagonal bracing configuration. It should be recognised that the shear forces in the diagonal braces in the vertical frames between legs are all in the same direction and, therefore, this bracing configuration does not meet the API Guidelines.

Figure 5.34 shows the load deflection curves for longitudinal Frame C, corresponding to environmental loading in both the positive and negative horizontal (x) directions. The positive case is considered first (40m-longtcp), where it can be seen that the load factor reaches a maximum value of 3.57. The first plastic hinge forms at a load factor of 3.19 at Joint Number 29 in Member (26-29), while the first component to fail is the tension brace (Member 26-29) in the bottom bay of the middle panel at a load factor of 3.19.

Figure 5.34 shows a rapid fall off in load (Point A) after the tension brace (Member 26-29) in the bottom bay of the middle panel and the compression brace (Member 24-29) in the bottom bay of the left panel fail. Very little energy is used by the structure in moving between Points A and B. The horizontal member just above the bracing in the bottom row (Member 25-26) and the compression brace in the bottom row of the right panel (member 26-31) acquire the load shed by the braces in the bottom bays of the left and middle panels and enable the overall structure residual strength to increase as the deflections proceed (between Points B and C).

A second fall off in load takes place (Point C) after the compression brace in the bottom bay of the right panel buckles (Member 26-31) and the horizontal Member (25-26) fails. The bracing in the upper rows and the legs acquire the load shed by the bracing in the bottom row and enable the overall structure residual strength to remain constant (Point D onwards) as the deflections proceed due to portal frame action in the lowest row.

For the negative loading case, the maximum load factors reached is 3.22. The first plastic hinge fails at a load factor of 3.21 at Joint Number 52 in Member (26-29), while the first component to fail is the compression brace, (Member 26-29) in the bottom bay of the middle panel, at a load factor of 3.26 after the ultimate load has been reached.

Figure 5.34 shows a fall off in load (Point E) after all the diagonal braces in the bottom row fail. Very little energy is used by the structure in deforming from the position corresponding to the ultimate strength to the first plateau (along Points F and G). The intermediate horizontal members and the bracing in the upper rows acquire the load shed by the bracing in the bottom row and enable the overall structure residual strength to remain constant as the deflection proceed (between Points F and G). A second fall off in load takes place (Point G) after members (20-25) and (21-25) start undergoing plastic deformations. The remaining legs, and the bracing in the top row, acquire the load shed by the bracing in the middle row and enable the overall structure residual strength to remain constant (beyond Point H) as the deflections proceed.

Examination of Figure 5.34 shows that there is a significant difference in the ultimate and post-ultimate response of the frame, depending on the direction of the applied loading. A loading applied to the frame in the positive horizontal direction exerts compressive forces on all the braces in the left and right panels, while a loading applied in the negative horizontal direction exerts compressive forces only on the braces in the middle panel. As tensile component behaviour tends to be more ductile than compressive behaviour, the 'weaker' ultimate and post-ultimate behaviour of the frame when subjected to a positive loading is to be expected.

Tables 5.5 and 5.6 present the dimensional and non-dimensional performance measures for the 40 metre longitudinal frames respectively.
Both measures of post-ultimate redundancy ($R_2$ and $R_4$) indicate that longitudinal Frame B and X and diagonal bracing has the highest degree of redundancy. Again it seems that a small margin of pre-ultimate redundancy is available. Three measures of pre-ultimate redundancy were used ($R_1$, $R_2$ and $R_3$). Results from both the 25 and 40 metre frames indicate that $R_3$ (load at ultimate / load at first component failure) is not an accurate measure of pre-ultimate redundancy because first component failure often occurs after the ultimate load has been reached. Furthermore very little difference exists between the load factors (and energies) corresponding to ultimate and first plastic hinge development. Therefore, for the 40 metre frames $R_1$ and $R_2$ are considered to be the most appropriate redundancy measures.

### Table 5.5

<table>
<thead>
<tr>
<th>Frame</th>
<th>Load at first plastic hinge</th>
<th>Load at first member failure</th>
<th>Ultimate load</th>
<th>Load at twice ultimate deflection</th>
<th>Energy at first member failure</th>
<th>Energy at ultimate</th>
<th>Energy at twice the ultimate deflection</th>
</tr>
</thead>
<tbody>
<tr>
<td>Longitudinal A +ve</td>
<td>3.16</td>
<td>3.17 (56C)</td>
<td>3.18</td>
<td>1.85</td>
<td>0.709</td>
<td>0.709</td>
<td>1.466</td>
</tr>
<tr>
<td>Longitudinal A -ve</td>
<td>3.16</td>
<td>3.18 (56T)</td>
<td>3.40</td>
<td>1.90</td>
<td>0.690</td>
<td>0.736</td>
<td>1.916</td>
</tr>
<tr>
<td>Longitudinal B</td>
<td>3.69</td>
<td>3.72 (56C)</td>
<td>3.78</td>
<td>3.10</td>
<td>0.886</td>
<td>0.910</td>
<td>2.426</td>
</tr>
<tr>
<td>Longitudinal C +ve</td>
<td>3.19</td>
<td>3.19 (55T)</td>
<td>3.57</td>
<td>2.2*</td>
<td>0.782</td>
<td>0.870</td>
<td>1.866*</td>
</tr>
<tr>
<td>Longitudinal C -ve</td>
<td>3.21</td>
<td>3.26</td>
<td>3.72</td>
<td>3.45</td>
<td>0.983</td>
<td>0.983</td>
<td>2.436</td>
</tr>
</tbody>
</table>

* estimate

### Table 5.6

<table>
<thead>
<tr>
<th>Frame</th>
<th>$R_1$</th>
<th>$R_2$</th>
<th>$R_3$</th>
<th>$R_4$</th>
<th>$R_5$</th>
<th>$R_6$</th>
<th>$R_7$</th>
<th>$R_8$</th>
<th>Ranking of post-ultimate redundancy</th>
</tr>
</thead>
<tbody>
<tr>
<td>Longitudinal A +ve</td>
<td>1.003</td>
<td>1.000</td>
<td>0.587</td>
<td>1.000</td>
<td>2.067</td>
<td>2007</td>
<td>30</td>
<td>3.0</td>
<td>3</td>
</tr>
<tr>
<td>Longitudinal A -ve</td>
<td>1.010</td>
<td>1.003</td>
<td>0.584</td>
<td>1.066</td>
<td>2.777</td>
<td>1708</td>
<td>38</td>
<td>3.12</td>
<td>1</td>
</tr>
<tr>
<td>Longitudinal B</td>
<td>1.020</td>
<td>1.016</td>
<td>0.818</td>
<td>1.027</td>
<td>2.738</td>
<td>1715</td>
<td>18</td>
<td>2.88</td>
<td>2</td>
</tr>
<tr>
<td>Longitudinal C +ve</td>
<td>1.120</td>
<td>1.120</td>
<td>0.704*</td>
<td>1.113</td>
<td>2.145*</td>
<td>1715</td>
<td>18</td>
<td>2.88</td>
<td>2</td>
</tr>
<tr>
<td>Longitudinal C -ve</td>
<td>1.000</td>
<td>0.988</td>
<td>0.930</td>
<td>1.000</td>
<td>2.478</td>
<td>1715</td>
<td>18</td>
<td>2.88</td>
<td>2</td>
</tr>
</tbody>
</table>

Notes:
- * estimate
- $R_1 =$ environmental load at ultimate / environmental load at first plastic hinge, $R_2 =$ environmental load at ultimate / environmental load at first component failure (RF), $R_3 =$ environmental load at ultimate / environmental load at first component failure (RF), $R_4 =$ environmental load at ultimate / energy at first member failure, $R_5 =$ energy at twice the ultimate deflection / energy at ultimate deflection, $R_6 =$ weight of jacket ($W_j$) (kN), $R_7 =$ fabrication cost (inc), $R_8 =$ reserve strength ratio (RSR).
Examination of Table 5.6 shows that all the frames have very close RSRs ($R_s$). However they have different margins of post ultimate redundancy and energy absorption capacity ($R_e$ and $R_s$). Frame B, which has the highest degree of post ultimate redundancy and energy absorption capacity, also has the highest fabrication cost associated with the number of brace associated welds at joints ($R_e$). However, it is interesting to note that it has the lowest weight. It seems that while X bracing configurations result in a higher cost associated with the additional number of brace-associated welds, it does not necessarily lead to a higher weight. Indeed, through careful design, the weight of an X-braced frame may be kept lower than that associated with diagonal or K-bracing.

5.4 80M DEEP FRAMES

Five 80 metre, two and four legged frames were considered. The bracing configurations are identical to those used for the 40 metre jackets. Transverse Frames A is shown in Figure 5.35.

The load deflection curve for transverse Frame A is shown in Figure 5.36, where it can be seen that the load reaches a maximum value of 3.21 (3.40 for the plastic hinge approach). The first plastic hinge develops at a load factor of 3.0 at Joint Number 22 in Member 22-15, while the first component to fail is the lower compression brace (Member 17-20) in the bottom bay at a load factor of 3.0. Again, it can be seen that the X-bracing configuration and the presence of horizontal members at plan framing levels allow the frame to carry a large proportion of its ultimate load after first component failure. Figure 5.36 shows a rapid fall off in load (Point A) after the bottom compression brace (Member 17-20) buckles and the bottom tension brace (Member 16-20) yields. Very little energy is used by the structure in deforming from the position corresponding to the ultimate strength (Point A) to the first plateau (ie. along B-C). The upper braces in the bottom bay and the bracing in the upper bays acquire the load shed by the lower braces in the bottom bay and enable the overall structure residual strength to remain constant as the deflections proceed (along B-C). A second fall off in load (Point C) takes place after the upper bracing in the bottom bay (Member (15-20) and (14-20)) undergo significant plastic deformation. The intermediate horizontal members and the braces in the upper bays acquire the load shed by the braces in the bottom bay and enable the overall structure residual strength to increase (between Points D and E). A third fall off in load (Point E) takes place with the failure of intermediate horizontal members and braces in the middle bay. The legs and bracing in the bay acquire the load shed by the bracing in the lower bays and enable the overall structure residual strength to increase as the deflections proceed due to portal frame action in the lower two bays. Transverse Frame B is shown in Figure 5.37, while the load deflection curve for Transverse Frame B is presented in Figure 5.38 where it can be seen that the load factor reaches a maximum value of 3.17 (3.05 for distributed plasticity). The first plastic hinge develops at a load factor of 2.90 at Joint Number 22 in Member 15-22, while the first component to fail is the lower compression brace (Member 19-15) in the middle bay at a load factor of 3.17. Comparison of Figure 5.36 and 5.38 confirms that the absence of horizontal members leads to a larger reduction in the post ultimate strength of the structure. Tables 5.7 and 5.8 present the dimensional and non-dimensional performance measures for the 80 metre transverse frames respectively.

Figure 5.38 shows a rapid fall of in load (Point A) after the lower compression brace in the middle bay (Member 15-19) buckles. Very little energy is used by the structure in deforming between the position corresponding to its ultimate load (Point A) and the first plateau (along Line B-C). The load is redistributed to braces in the lower and middle bays which enable the overall structure residual strength to remain constant while the deformations increase slightly (along Line B-C). A second rapid fall off in load (Point C) takes place after the lower
compression brace in the bottom bay buckles. Again very little energy is used by the structure in deforming between Point C and the second plateau (along Line D-E). The legs and the bracing in the top bay acquire the load shed by the braces in the lower bays and enable the overall structural residual strength to remain constant as the deflections proceed (along Line D-E). A third rapid fall off in load (Point E) takes place after the compression brace in the upper bay buckles. The legs acquire the load shed by the braces and enable the overall structure residual strength to remain constant as the deflections proceed due to portal frame action in the jacket legs.

Table 5.7

<table>
<thead>
<tr>
<th>Frame</th>
<th>Load at first plastic hinge</th>
<th>Load at first member failure</th>
<th>Ultimate load (Ph/dP)</th>
<th>Load at twice ultimate deflection</th>
<th>Energy at first member failure</th>
<th>Energy at ultimate</th>
<th>Energy at twice the ultimate deflection</th>
</tr>
</thead>
<tbody>
<tr>
<td>Transverse A</td>
<td>3.0</td>
<td>3.40</td>
<td>3.40/3.21</td>
<td>1.9</td>
<td>0.585</td>
<td>0.626</td>
<td>1.591</td>
</tr>
<tr>
<td>Transverse B</td>
<td>2.9</td>
<td>3.17</td>
<td>3.17/3.05</td>
<td>0.96</td>
<td>0.566</td>
<td>0.595</td>
<td>0.961</td>
</tr>
</tbody>
</table>

Table 5.8

Non-dimensional performance measures and ranking for the 80 metre transverse frames

<table>
<thead>
<tr>
<th>Frame</th>
<th>R₁</th>
<th>R₂</th>
<th>R₃</th>
<th>R₄</th>
<th>R₅</th>
<th>R₆</th>
<th>R₇</th>
<th>R₈</th>
<th>Ranking of pre-ultimate redundancy</th>
<th>Ranking of post-ultimate redundancy</th>
</tr>
</thead>
<tbody>
<tr>
<td>Transverse A</td>
<td>1.130</td>
<td>1.0</td>
<td>0.592</td>
<td>1.070</td>
<td>2.541</td>
<td>5713</td>
<td>16</td>
<td>2.86</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Transverse B</td>
<td>1.093</td>
<td>1.0</td>
<td>0.315</td>
<td>1.051</td>
<td>1.615</td>
<td>5668</td>
<td>12</td>
<td>2.69</td>
<td>2</td>
<td>2</td>
</tr>
</tbody>
</table>

R₁ = environmental load at ultimate / environmental load at first plastic hinge, R₂ = environmental load at ultimate / environmental load at first component failure (RF), R₃ = environmental load at twice the ultimate deflection / environmental load at ultimate (RS), R₄ = energy at environmental ultimate load / energy at first member failure, R₅ = energy at twice the ultimate deflection / energy at environmental ultimate load, R₆ = weight of jacket (Wₖ) (kN), R₇ = fabrication cost (toa), R₈ = reserve strength ratio (RSR).

Both measures of post ultimate redundancy (R₆ and R₈) indicate that Transverse Frame A with X bracing and horizontal components at plan bracing level has the highest degree of redundancy. Due to the proximity of first member failure and the ultimate load, R₆ is not an appropriate measure of pre-ultimate redundancy. However, both R₇ and R₈ reveal that Transverse Frame A has a higher degree of pre-ultimate redundancy.

Examination of Table 5.8 shows that both frames have very close RSRs (R₆), however they have significantly different margins of pre and post ultimate redundancy and energy absorption capacity. In both cases Frame A has a higher degree of redundancy. The cost, expressed in terms of the weight of the jacket (R₆) and the cost of fabrication (R₇), is also higher for Frame A.
Figure 5.39 shows the unsymmetric four-legged longitudinal Frame A with K and diagonal bracing. The bracing configuration is similar to that used for the four legged 40m deep longitudinal Frame A. The load deflection curves for longitudinal Frame A corresponding to environmental loading in both the positive and the negative horizontal (x) directions are shown in Figure 5.40. The positive case (80m-longtap) is considered first, where it can be seen that the load factor reaches a maximum value of 2.0. The first plastic hinge develops at a load factor of 1.1 at Joint Number 42 in Member (28-42), while the first component to develop significant plasticity is Member (28-42) at a load factor of 1.8.

Figure 5.40 shows a nonlinear behaviour at early stages of the jacket response. The load reaches a plateau after several of the horizontal members (Members (24-41), (28-42) and (40-27) and bracing in the lower bay (Member 24-42) develop plastic deformations. The legs acquire the load shed by the braces and horizontal members and enable the overall residual structure response to remain constant as the deflections proceed due to portal frame action in the legs.

The case for the negative loading (80m-longtan) is considered next, where it can be seen that the maximum load factor reached is 2.05. The first plastic hinge develops at a load factor of 1.2 in Joint Number 43 in Member (43-31). Figure 5.40 indicates that the load reaches a plateau after several horizontal members develop significant plasticity (Member 24-41), (39-23), (43-31) and (40-27). The legs and the braces acquire the load shed by the horizontal members. A more severe drop in load occurs after the diagonal brace (Member 43-27) develops significant plasticity.

Figure 5.41 shows longitudinal Frame B, which has X and diagonal bracing. The bracing configuration in this frame is identical to that used in the 40m longitudinal Frame B. Both frames satisfy all API ductility requirements.

The load deflection curve for Frame B is shown in Figure 5.42, where it can be seen that the load factor reaches a maximum value of 4.03. The first plastic hinge develops at a load factor of 2.73 at Joint Number 35 in Member (35-27), while the first component to fail is the bottom compression brace in the middle bay of the middle panel (Member 26-39) at a load factor of 3.66.

Figure 5.42 shows a rapid fall off in load (Point A) after different braces and members in the lowest row develop plastic deformation. The next increment in load (along Line B-C) corresponds to some of the braces in the bottom row acquiring more load. However, as more plastic deformation occurs, a second fall off in load (Point C) takes place. The legs and the upper braces acquire the load and enable the overall structure residual strength to reach a constant value.

Finally, Figure 5.43 shows longitudinal Frame C, which is similar to the 40m longitudinal Frame C and has an unsymmetric diagonal bracing configuration. Figure 5.44 shows the load deflection curves for longitudinal Frame C, corresponding to environmental loading in both the positive and negative horizontal (x) directions. The positive case (80m-longtcp) is considered first, where it can be seen that the load factor reaches a maximum value of 3.54.

Figure 5.44 shows a rapid fall off in load (Point A) after the compression braces in the bottom row (Members (24-29) and (33-26), the compression brace in the middle bay of the left panel Member (20-25) and the intermediate horizontal component Member (24-25) undergo significant plastic deformation. The intermediate horizontal components (Members (20-21), (21-22) and (25-26) acquire the load shed by the diagonal braces and enable the overall
structure residual strength to remain constant as the deflections proceed (along Line B-C). Another fall off in load (Point C) takes place after the intermediate horizontal members lose their carrying capacity due to the development of significant plastic deformation.

Figure 5.44 also shows the load deflection curve for the negative case (80m-longtcdn), where it can be seen that the maximum load factor reached is 3.56. The first plastic hinge develops at a load factor of 3.05, while first component failure occurs in the bottom tension brace in the right panel at a load factor of 3.56.

Figure 5.44 shows a rapid drop in the load (Point D) after the bottom tension brace (Member 31-26) in the right panel fails. The upper braces and intermediate horizontal members acquire the load shed by the lower tension brace and enable the overall structure residual strength to remain constant as the deflections proceed (along E-F). A second fall off in load (Point F) occurs after the upper tension brace in the right panel fails.

<table>
<thead>
<tr>
<th>Frame</th>
<th>Load at first plastic hinge</th>
<th>Load at first member failure</th>
<th>Ultimate load</th>
<th>Load at twice ultimate deflection</th>
<th>Energy at first member failure</th>
<th>Energy at ultimate</th>
<th>Energy at twice the ultimate deflection</th>
</tr>
</thead>
<tbody>
<tr>
<td>Longitudinal A+ve</td>
<td>N/A</td>
<td>1.8</td>
<td>N/A</td>
<td>2.0</td>
<td>1.4</td>
<td>0.37</td>
<td>0.4</td>
</tr>
<tr>
<td>Longitudinal A-ve</td>
<td>N/A</td>
<td>1.75</td>
<td>N/A</td>
<td>2.10</td>
<td>1.6*</td>
<td>0.37</td>
<td>0.45</td>
</tr>
<tr>
<td>Longitudinal B</td>
<td>2.73</td>
<td>3.66</td>
<td>4.13</td>
<td>4.43</td>
<td>3.0</td>
<td>0.775</td>
<td>1.048</td>
</tr>
<tr>
<td>Longitudinal C+ve</td>
<td>2.20</td>
<td>3.40</td>
<td>5.56</td>
<td>5.00</td>
<td>2.0*</td>
<td>0.788</td>
<td>1.758</td>
</tr>
<tr>
<td>Longitudinal C-ve</td>
<td>3.10</td>
<td>3.78</td>
<td>3.78</td>
<td>3.54</td>
<td>1.13</td>
<td>0.756</td>
<td>0.859</td>
</tr>
</tbody>
</table>

* estimate
### Table 5.10
Non-dimensional performance measures and ranking for the 80 metre longitudinal frames

<table>
<thead>
<tr>
<th>Frame</th>
<th>$R_1$</th>
<th>$R_2$</th>
<th>$R_3$</th>
<th>$R_4$</th>
<th>$R_5$</th>
<th>$R_6$</th>
<th>$R_7$</th>
<th>$R_8$</th>
<th>Ranking of pre-ultimate redundancy</th>
<th>Ranking of post-ultimate redundancy</th>
</tr>
</thead>
<tbody>
<tr>
<td>Longitudinal A +ve</td>
<td>N/A</td>
<td>1.11</td>
<td>0.7*</td>
<td>1.08</td>
<td>2.125 *</td>
<td>21424</td>
<td>30.15</td>
<td>1.65</td>
<td>3</td>
<td>2</td>
</tr>
<tr>
<td>Longitudinal A -ve</td>
<td>N/A</td>
<td>1.2</td>
<td>0.78*</td>
<td>1.22</td>
<td>2.0*</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Longitudinal B</td>
<td>1.48</td>
<td>1.21</td>
<td>0.744</td>
<td>1.352</td>
<td>2.276</td>
<td>12784</td>
<td>38.34</td>
<td>3.34</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Longitudinal C +ve</td>
<td>2.23</td>
<td>1.63</td>
<td>0.407 *</td>
<td>2.232</td>
<td>2.86*</td>
<td>12674</td>
<td>18</td>
<td>3.80</td>
<td>2</td>
<td>3</td>
</tr>
<tr>
<td>Longitudinal C -ve</td>
<td>1.12</td>
<td>1.00</td>
<td>0.319</td>
<td>2.124</td>
<td>1.884</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Notes:
* estimate
$R_1$ = environmental load at ultimate / environmental load at first plastic hinge, $R_2$ = environmental load at ultimate / environmental load at first component failure (RF), $R_3$ = environmental load at twice the ultimate deflection / environmental load at ultimate (RS), $R_4$ = energy at environmental ultimate load / energy at first member failure, $R_5$ = energy at twice the ultimate deflection / energy at environmental ultimate load, $R_6$ = weight of jacket (WJ) (kN), $R_7$ = fabrication cost (f2k), $R_8$ = reserve strength ratio (RSR).

### 5.5 100M DEEP FRAMES

Three 100 metre, 2 two-legged and 1 four-legged frames were considered. Transverse Frame A, which is shown in Figure 5.45, is a four bay, two-legged structure with X-bracing in the top 3 bays and K-bracing in the bottom bay while the load deflection curve is presented in Figure 5.46 where it can be seen that the load factor reaches a maximum value of 3.04, Figure 5.46 shows a rapid fall off in load (Point A) after the compression members in the second and third bays (Members (17-22), (22-14) and (15-21)), together with the bottom horizontal member (Member 19-23) fail. A second and sharper fall off in load (Point B) occurs after the remaining bracing and the intermediate horizontal members fail. Transverse Frame B is shown in Figure 5.47, has K-bracing in all bays. Figure 5.48 shows the load-deflection curve for transverse Frame B, where it can be seen that the load factor reaches a maximum of 3.21. Figure 5.48 shows a rapid fall off in load (Point A) after the horizontal component in the second bay (Member 21-15) fails. A second fall off in load (Point B) occurs after the horizontal component Member (23-18) fails. The longitudinal frame, shown in Figure 5.49, has a diagonal bracing configuration which does not satisfy the API guidelines. The load deflection curves for the 100m deep longitudinal frame corresponding to environmental loading in the positive and negative horizontal (x) directions are shown in Figures 5.50 and 5.1 respectively. The positive case (100m-longtap) is considered first. Figure 5.50 shows the rapid fall off in load (Point A) after the lowest horizontal component (Member 38-39) in the right panel fails. The legs and braces in the upper bays acquire the load shed by the collapsed member and enable the structure residual strength to increase as the deflections proceed (along Line B-C). A second fall off in load (Point C) takes place after the tension brace in the
second bay fails (Member 21-24). Again, the legs and the remaining braces acquire the additional load and enable the overall structure residual strength to increase (Point D onwards) as the deflections proceed.

Figure 5.51 shows the load deflection curve for the negative loading case where it can be seen that the load factor reaches a maximum value of 4.03. Figure 5.51 also shows the rapid fall off in load (Point A) after the tension brace in the second from bottom bay of the right panel yields (Member 31-65).

The intermediate horizontal members and the remaining braces acquire the load shed by the collapsed member and enable the overall structure response to increase (along Line B-C) and reach a constant value (Point C) as the deflections proceed.

Tables 5.11 and 5.12 present the dimensional and non-dimensional performance measures. Again, both post-ultimate performance measures show that the X bracing configuration adopted in Frame A results in the highest degree of redundancy. The only available pre-ultimate measure also shows that Frame A has a higher degree of redundancy.

### Table 5.11

**Performance measures for the 100 metre transverse and longitudinal frames**

<table>
<thead>
<tr>
<th>Frame</th>
<th>Load at first plastic hinge</th>
<th>Load at first member failure</th>
<th>Ultimate load (PH)</th>
<th>Ultimate load (DP)</th>
<th>Load at twice ultimate deflection</th>
<th>Energy at first member failure</th>
<th>Energy at ultimate</th>
<th>Energy at twice the ultimate deflection</th>
</tr>
</thead>
<tbody>
<tr>
<td>Transverse A</td>
<td>2.59</td>
<td>after ultimate</td>
<td>2.60</td>
<td>3.04</td>
<td>1.2</td>
<td>n/a</td>
<td>1.0</td>
<td>1.713</td>
</tr>
<tr>
<td>Transverse B</td>
<td>2.62</td>
<td>after ultimate</td>
<td>2.62</td>
<td>3.21</td>
<td>1.0</td>
<td>n/a</td>
<td>1.0</td>
<td>1.677</td>
</tr>
<tr>
<td>Longitudinal A+ve</td>
<td>3.65</td>
<td>3.84</td>
<td>4.31</td>
<td>3.99</td>
<td>2.5*</td>
<td>1.369</td>
<td>1.496</td>
<td>3.756*</td>
</tr>
<tr>
<td>Longitudinal A-ve</td>
<td>3.08</td>
<td>4.12</td>
<td>4.13</td>
<td>4.03</td>
<td>2.5*</td>
<td>1.186</td>
<td>1.552</td>
<td>3.198*</td>
</tr>
</tbody>
</table>

* estimate

Examination of Table 5.12 shows that both transverse frames have very close RSR values (R8), however they have significantly different margins of pre-ultimate and post ultimate redundancy and energy absorption capacity. In both cases, Frame A (X and K bracing) has higher margins. Again it can be seen that while Frame A has a higher fabrication cost, it does not have a higher weight.
Table 5.12
Non-dimensional performance measures for the 100 metre transverse and longitudinal frames

<table>
<thead>
<tr>
<th>Frame / load</th>
<th>$R_1$</th>
<th>$R_2$</th>
<th>$R_3$</th>
<th>$R_4$</th>
<th>$R_5$</th>
<th>$R_6$</th>
<th>Ranking of pre-ultimate redundancy</th>
<th>Ranking of post-ultimate redundancy</th>
</tr>
</thead>
<tbody>
<tr>
<td>Transverse A</td>
<td>1.003</td>
<td>&lt;1.0</td>
<td>0.394</td>
<td>N/A</td>
<td>1.977</td>
<td>5757</td>
<td>22</td>
<td>2</td>
</tr>
<tr>
<td>Transverse B</td>
<td>1.000</td>
<td>&lt;1.0</td>
<td>0.311</td>
<td>N/A</td>
<td>1.394</td>
<td>6152</td>
<td>16</td>
<td>1</td>
</tr>
<tr>
<td>Longitudinal A +ve</td>
<td>1.180</td>
<td>1.12</td>
<td>0.626*</td>
<td>1.093</td>
<td>2.510*</td>
<td>15901</td>
<td>24</td>
<td>2</td>
</tr>
<tr>
<td>Longitudinal A -ve</td>
<td>1.340</td>
<td>1.002</td>
<td>0.622*</td>
<td>1.308</td>
<td>2.061*</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Notes:

* estimate

$R_1 =$ environmental load at ultimate / environmental load at first plastic hinge, $R_2 =$ environmental load at ultimate / environmental load at first component failure (RF), $R_3 =$ environmental load at twice the ultimate deflection / environmental load at ultimate (RS), $R_4 =$ energy at environmental ultimate load / energy at first member failure, $R_5 =$ energy at twice the ultimate deflection / energy at environmental ultimate load, $W =$ weight of jacket (kN), $C =$ fabrication cost (n), $R_s =$ reserve strength ratio (RSR).

5.6 140M DEEP FRAMES

Two 140 metre, four legged frames were considered. Frame A, which is shown in Figure 5.52, satisfies the API ductility recommendations, while Frame B, which is shown in Figure 5.54, does not. Therefore, it is expected that Frame A will have higher pre-ultimate and post-ultimate performance measures.

Figure 5.53 shows the load deflection curves for the 140m deep longitudinal Frame A, corresponding to wave loading in both the positive and the negative horizontal (x) directions. The positive case (140m-longtap) is considered first, where it can be seen that the load factor reaches a maximum value of 3.20. Figure 5.53 shows a reduction in stiffness (Point A) after a lower horizontal member undergoes plastic deformation (Member 42-43). The braces and the legs acquire the load shed by the lower leg portion and enable the overall structure residual strength to increase as the deflections proceed along (Line A-B). A gradual fall off in load (Point B) takes place after the horizontal Member (40-41) fails.

The negative case (140m-longtan) is considered next, where it can be seen that the load factor reaches a maximum value of 3.01. Figure 5.53 shows a drop in load (Point C) after Members (31-35) and (35-39) located on the right jacket leg, horizontal members (42-43) and (34-35) undergo significant plastic deformation. The solution does not proceed due to numerical instability problems.

Figure 5.54 shows the 140m longitudinal Frame B which has a diagonal bracing configuration that does not satisfy the API ductility requirements. The load deflection curves, corresponding to environmental loading in both the negative and the positive horizontal (x) directions are shown in Figure 5.55. The positive case (140m-longbh) is considered first, where it can be seen that the load factor reaches a maximum value at it's initial peak equal to 4.10. Figure 5.55
shows a reduction in stiffness (Point A) after the bottom compression brace in the right panel fails. The legs and remaining braces acquire the load shed by the compression brace and enable the overall structure residual strength to remain constant as the deflections proceed.

The negative loading case is considered next, where it can be seen that the load factor reaches a maximum value of 4.35. Figure 5.55 shows a rapid fall off in load (Point B) after the compression brace in the second row of the left panel fails (Member 24-21). The legs and the remaining braces acquire the load shed by the compression brace and enable the overall structure residual strength to increase as the deflections proceed (along Line C-D). Numerical instability problems leads to a divergence beyond Point D.

Table 5.13 and 5.14 present the dimensional and non-dimensional performance measures. Both post-ultimate performance measures show that Frame A which satisfies API ductility requirements has a higher degree of redundancy. Of the three pre-ultimate performance measures \( R_1 \) (energy at ultimate + energy at first plastic hinge) results in the same degree of ranking.

<table>
<thead>
<tr>
<th>Frame / load</th>
<th>load at first plastic hinge</th>
<th>Load at first member failure</th>
<th>Ultimate load PH</th>
<th>Ultimate load DP</th>
<th>Load at twice ultimate deflection</th>
<th>Energy at first member failure</th>
<th>Energy at ultimate</th>
<th>Energy at twice the ultimate deflection</th>
</tr>
</thead>
<tbody>
<tr>
<td>Longitudinal A +ve</td>
<td>2.65</td>
<td>3.30</td>
<td>3.30</td>
<td>3.20</td>
<td>1.2</td>
<td>1.153</td>
<td>1.349</td>
<td>3.151</td>
</tr>
<tr>
<td>Longitudinal A -ve</td>
<td>2.65</td>
<td>3.11</td>
<td>3.33</td>
<td>3.01</td>
<td>1.7*</td>
<td>1.550</td>
<td>1.768</td>
<td>3.768*</td>
</tr>
<tr>
<td>Longitudinal B +ve</td>
<td>3.73</td>
<td>4.26</td>
<td>4.31</td>
<td>4.10</td>
<td>1.4*</td>
<td>1.540</td>
<td>1.763</td>
<td>4.773*</td>
</tr>
<tr>
<td>Longitudinal B -ve</td>
<td>3.86</td>
<td>4.00</td>
<td>4.43</td>
<td>4.35</td>
<td>1.2*</td>
<td>1.990</td>
<td>2.063</td>
<td>5.353*</td>
</tr>
</tbody>
</table>

* estimate

Examination of Table 5.14 shows that both frames have very close values of \( R_4 \), however they have different margins of pre and post ultimate redundancy and energy absorption capacity. Both frames have diagonal bracing configurations and therefore have similar \( R_4 \) ratios.
Table 5.14
Non-dimensional performance measures for the 140 metre longitudinal frames

<table>
<thead>
<tr>
<th>Frame / load</th>
<th>$R_1$</th>
<th>$R_2$</th>
<th>$R_3$</th>
<th>$R_4$</th>
<th>$R_5$</th>
<th>$R_6$</th>
<th>$R_7$</th>
<th>$R_8$</th>
<th>Ranking of pre-ultimate redundancy</th>
<th>Ranking of post-ultimate redundancy</th>
</tr>
</thead>
<tbody>
<tr>
<td>Longitudinal A $\pm$ve</td>
<td>1.245</td>
<td>1.00</td>
<td>0.387</td>
<td>1.169</td>
<td>2.336</td>
<td></td>
<td></td>
<td></td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Longitudinal A $\pm$ve</td>
<td>1.256</td>
<td>1.07</td>
<td>0.564*</td>
<td>1.141</td>
<td>2.131*</td>
<td>18920</td>
<td>30</td>
<td>3.01</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Longitudinal B $\pm$ve</td>
<td>1.150</td>
<td>1.011</td>
<td>0.341*</td>
<td>1.145</td>
<td>2.707*</td>
<td></td>
<td></td>
<td></td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>Longitudinal B $\pm$ve</td>
<td>1.48</td>
<td>1.108</td>
<td>0.273*</td>
<td>1.037</td>
<td>2.594*</td>
<td>22209</td>
<td>30</td>
<td>3.10</td>
<td>2</td>
<td>2</td>
</tr>
</tbody>
</table>

Notes:
* estimate
$R_1 =$ environmental load at ultimate / environmental load at first plastic hinge, $R_2 =$ environmental load at ultimate / environmental load at first component failure (RF), $R_3 =$ environmental load at twice the ultimate deflection / environmental load at ultimate (RS), $R_4 =$ energy at environmental ultimate load / energy at first member failure, $R_5 =$ energy at twice the ultimate deflection / energy at environmental ultimate load, $R_6 =$ weight of jacket ($W_{ij}$) (kN), $R_7 =$ fabrication cost (AED), $R_8 =$ reserve strength ratio (RSR).
Figure 5.1
Computer plot of 25m transverse / longitudinal Frame A

Figure 5.2
Load deflection curve of 25m Frame A
Figure 5.3
Member force variation in bottom bay of 25m Frame A

Figure 5.4
Member force variation in top bay of 25m Frame A
Figure 5.5
Computer plot of 25m transverse / longitudinal Frame B

Figure 5.6
Load deflection curves of 25m Frame B
Figure 5.7
Member force variation in 25m Frame B +ve load case

Figure 5.8
Member force variation in 25m frame B -ve loading case
Figure 5.9
Computer plot of 25m transverse / longitudinal Frame C

Figure 5.10
Computer plot of 25m transverse / longitudinal Frame D

Figure 5.11
Load deflection curves of 25m Frames C and D
Figure 5.12
Member force variation in bottom bay of 25m Frame C

Figure 5.13
Member force variation in top bay of 25m Frame C
Figure 5.14
Member force variation in bottom bay of 25m Frame D

Figure 5.15
Member force variation in top bay of 25m Frame D
Figure 5.16
Computer plot of 25m transverse / longitudinal Frame E

Figure 5.17
Load deflection curves of 25m Frame E
Figure 5.18
Member force variation in 25m Frame E -ve loading case
Figure 5.19
Computer plot of 40m transverse Frame A

Figure 5.20
Load deflection curve of 40m deep transverse Frame A
Figure 5.21
Computer plot of 40m transverse Frame B

Figure 5.22
Load deflection of 40m deep transverse Frame B
Figure 5.23
Computer plot of 40m longitudinal Frame A

Figure 5.24
Load deflection curve for 40m deep longitudinal Frame A - positive loading

Figure 5.25
Load deflection curve for 40m deep longitudinal Frame A - negative loading
Figure 5.26
Computer plot of 40m longitudinal Frame B

Figure 5.27
Load deflection curve of 40m deep longitudinal Frame B
Figure 5.28
Member force variation in bottom bay of left panel in 40m longitudinal Frame B

Figure 5.29
Member force variation in bottom bay of middle panel in 40m longitudinal Frame B
Figure 5.30
Member force variation in bottom bay of right panel in 40m longitudinal Frame B

Figure 5.31
Member force variation in middle bay of middle panel in 40m deep longitudinal Frame B

Figure 5.32
Member forces in middle and top bays of the left and right panels in 40m deep longitudinal Frame B
Figure 5.33
Computer plot of 40m longitudinal Frame C

Figure 5.34
Load deflection curve for 40m deep longitudinal Frame C
Figure 5.35
Computer plot of 80m deep transverse Frame A

Figure 5.36
Load deflection curve for 80m deep transverse Frame A
Figure 5.37
Computer plots of 80m deep transverse Frame B

Frame 5.38
Load deflection curve for 80m deep transverse Frame B
Figure 5.39
Computer plot of 80m deep longitudinal Frame A

Figure 5.40
Load deflection curve for 80m deep longitudinal Frame A
Figure 5.41
Computer plot of 80m longitudinal Frame B

Figure 5.42
Load deflection curves for 80m deep longitudinal Frame B
Figure 5.43
Load deflection curves of 80m deep longitudinal Frame C

Figure 5.44
Load deflection curves of 80m longitudinal Frame C
Figure 5.45
Computer plots of 100m deep transverse Frame A

Figure 5.46
Load deflection curve for 100m transverse Frame A
Figure 5.47
Computer plot of 100m deep transverse Frame B

Figure 5.48
Load deflection curve for 100m transverse Frame B
Figure 5.49
Computer plot of the 100m deep longitudinal frame

Figure 5.50
Load deflection curves of the 100m longitudinal frame - positive loading

Figure 5.51
Load deflection curves for the 100m longitudinal frame - negative loading
Figure 5.52
Computer plot of 140m deep longitudinal Frame A

Figure 5.53
Load deflection curves for 140m longitudinal Frame A
Figure 5.54
Load deflection curves for 140m longitudinal Frame B

Figure 5.55
Load deflection curves for 140m longitudinal Frame B
6. RESULTS OF SENSITIVITY STUDY

Pushover analyses are often carried out with certain parameters, such as loading conditions and material properties, held constant. These parameters are often assumed to have second order effects on the response and the effects of their variation are therefore not considered in the analysis. The purpose of this section is to explore if, and when, the effects of the variation in certain loading and structural parameters may be neglected.

In analysing frames to represent offshore jackets, an important parameter is the stress-strain steel characteristics in relation to 'typical' offshore materials. Another factor is the influence of initial imperfections on the ultimate strength and post-ultimate behaviour of jackets. The proportion of dead load to environmental load is one of the variations identified as a result of the survey of North Sea structures (reported in Chapter 3). To study the influence of the above factors on the performance measures obtained from ultimate strength analyses, deterministic sensitivity studies were carried out in relation to the following parameters:

- Material behaviour, in particular strain hardening effects.
- Initial imperfections.
- Distribution of hydrodynamic loading.
- Joint flexibility and strength.

To this end, sensitivity studies on initial imperfections, distribution of hydrodynamic loading, joint flexibility and material behaviour were carried out.

The results of these studies are presented in the following sections, with a detailed discussion in Section 7.

6.1 SENSITIVITY STUDY ON INITIAL IMPERFECTIONS

A total of twenty-six analyses were carried out to determine the effects of initial imperfections on the response of jackets. In all cases, initial imperfections were applied to the braces in compression. The maximum magnitude of the applied initial imperfections (at the middle of the member) varied between L/500 and L/2000, where L is the brace length. The tolerance on the out-of-straightness in the braces of L/1200 where L is the member length, as defined in Section 6.2.1.10 of EEMUA Construction Specification Guidelines, lies within the above range of initial imperfections. The direction of the initial imperfections was also varied and a positive sign indicates that the imperfections are in the direction of the member deflection.

Figure 6.1 shows the load deflection curves for the 25m deep transverse Frame A, with different imperfection profiles applied to the compression braces. It can be seen that a positive imperfection results in a slight decrease in the post ultimate strength of the jacket and vice-versa. However, there is no significant difference in the ultimate strength of the jacket. This may be explained by the fact that, as discussed in Section 5.1, the bottom compression brace is the first member to fail after the ultimate load has been reached. The same trend can be seen in Figure 6.2, which shows the load deflection curves for the 25m deep Frame C.

For imperfection magnitudes in the range of the recommended code tolerances (L/1000 and L/2000) the effects of initial imperfections on the ultimate response was found to be insignificant. However, as the imperfection magnitude increases (L/500 in Figure 6.2) the
effect of initial imperfections on the response becomes more significant. These observations confirm the validity of the code recommendations regarding the maximum allowable tolerances.

Therefore, it is recommended that the effects of initial imperfections may be neglected in the case of intact structures that have been designed to recognise and accept standards and subsequently fabricated with approved materials to sound quality procedures. However for structure where one or more of the steps in the construction process has not been fully verified, the effects of initial imperfections on the ultimate and post ultimate structural response should be examined.

Ship impact analysis, which are often carried out by the offshore industry, are concerned with gross structural damage. However, there is a level of damage below the one described above (eg. supply boat impact of a bracing component) which must also be considered. For critical members which may have a significant risk of damage, eg. bracing components through the splash zone, a larger magnitude of imperfections may be considered as a means to assess the sensitivity of the structure to such impact damage.

In such cases, magnitudes of imperfections twice or three time the order of the maximum allowable tolerances are not considered excessive. For example a 20 metre component may easily sustain a deformation of 40mm (more than twice the maximum allowable tolerance) due to impact.

6.2 SENSITIVITY STUDY ON DISTRIBUTION OF STILL WATER TO HYDRODYNAMIC LOADING

A total of six analyses were carried out to determine the effects of the distribution of the hydrodynamic loading on the response of jackets. As mentioned in Section 3, the SACS program was used to generate the loading conditions corresponding to the still water case and the 100-year storm. These loading conditions were then applied as equivalent nodal loads in pushover analyses. To vary the distribution of the hydrodynamic loading, the proportion of the still water load (SWL) to the storm load was changed. Starting from the base analyses, three additional analyses were carried out using a value of 0.5 SWL, 1.5 SWL and 2.0 SWL for two frames.

Figures 6.3 and 6.4 show the load deflection curves, using the above hydrodynamic loading distributions, for the 25m deep Frame C and the 40m deep longitudinal Frame B respectively.

In both cases, there is no significant difference in the results. This is due to the fact that the environmental loads will continue to dominate the failure mode until a much higher ratio of still water to environmental loading is reached.

6.3 SENSITIVITY STUDY ON JOINT FLEXIBILITY

The effects of joint flexibility and strength on the response of steel jackets lie outside the scope of this study. It has been investigated by various research groups, including the Joint Industry Project carried out by BOMEL (Bolt et al 1994 a and b). In this study, few cases will be considered to demonstrate the effects of joint flexibility and strength on the response.
A total of three analyses were carried out to demonstrate the effects of joint flexibility on the response of jackets. As mentioned in Section 4, five-part force-displacement springs were used to model the flexible joints. Figures 6.5 and 6.6 show the nonlinear springs used to model the behaviour of X joints. Throughout the report compressive forces and stresses are denoted as negative. The behaviour of X-joints in compression is characterised by a four part curve. The joint behaviour is linear elastic up to a load $P_{\alpha}$ corresponding to the displacement $\delta_{\alpha}$ when the brace starts ovaling. Under additional loading, the force ($P_{\alpha}$) is kept constant while the displacement increases up to a value of $\delta_{\alpha}$ corresponding to contact between the brace walls adjacent to the joint. The contact between the brace walls then stiffens the joint, which can now carry an additional load up to a total value of $P_{\alpha}$ corresponding to a displacement $\delta_{\alpha}$ when the members fail in compression. If the load is increased further, the force is kept constant at a value of $P_{\alpha}$ while the displacement keeps on increasing due to the loss of stiffness. There is a lack of data to justify the last part of the compression curve which describes the behaviour after the value of $P_{\alpha}$ is reached.

The tensile behaviour of the X joint behaviour is linear elastic up to a force value $P_{\gamma}$ which corresponds to yielding in the outer fibres of the brace cross-section adjacent to the joint. Under additional loading, the brace undergoes larger deformations, due to reduction in stiffness, up to a force value of $P_{\gamma}$ corresponding to a displacement $\delta_{\gamma}$. If the load is increased further, the force in the joint is limited to the ultimate force due to the loss of stiffness.

Figure 6.7 shows the load deflection curve for 25m Frame C, with flexible and rigid joint behaviour. It can be seen that assuming rigid joint behaviour may lead to an upper bound on the response. Figure 6.8 shows the axial force variation in the joint springs with the load; where it can be seen that both joints have reached their tensile yield strength. The above discussion and examples demonstrate that joint flexibility and strength have significant effects on the ultimate and post-ultimate response of steel jackets and should therefore be accounted for when carrying out pushover analyses.

### 6.4 SENSITIVITY STUDY ON MATERIAL BEHAVIOUR

A total of twelve analyses were carried out to investigate the effects of strain hardening on the response of jackets. As mentioned in Section 4, Type 32 elements with a distributed plasticity formulation are used to model strain hardening. Strain hardening factors (which define the post-yield slope of the stress-strain curve) of 1% and 2% were considered.

Figure 6.9 shows the load deflection curves for the 25m deep Frame A, using a variety of strain hardening values. It can be seen that the ultimate strength is not affected by the value of strain hardening. However, the post-ultimate behaviour is slightly affected with the frame responding in a stiffer manner as a value of strain hardening increases.

A different type of behaviour is observed in Figure 6.10 which shows the load deflection curve for the 25m deep Frame C. In this case the load corresponding to the first peak (Point A) remains unaffected by the value of the strain hardening parameter, however as yielding develops in various components differences of up to 40% are recorded in the ultimate response corresponding to strain hardening parameters of 2% and 0.1%. Figure 6.11 shows the load deflection curve for the 80m deep longitudinal Frame B, where it can be seen that the ultimate response is unaffected by the value of the strain hardening parameter. However, as seen in the case of the 25m deep Frame A, the post-ultimate behaviour is moderately affected with a maximum difference of 17% recorded at a displacement of 1.0m.
The above discussion and examples demonstrate that the choice of the material behaviour, in this case the value of the strain hardening parameter, may have significant effects on the ultimate and post-ultimate response of steel jackets.

The effects of variations in the material behaviour on the ultimate and post ultimate response of jackets should therefore be accounted for when carrying out pushover analyses.

6.5 CONCLUDING REMARKS

The effects of variations in various parameters which are usually held constant when carrying out pushover analyses have been examined. In particular, variations in the magnitude of initial imperfections and strain hardening parameter, the proportion of still water to environmental loads and the modelling of joint flexibility and strength, were considered. Results from pushover analyses demonstrate that variations in any of the above parameters may significantly alter the ultimate and post-ultimate response of steel jackets. Therefore, it is recommended that effects of variations in these parameters should be considered and, if significant, accounted for when carrying out pushover analyses.
Figure 6.1
Effect of initial imperfections on the behaviour of the 25m deep Frame A

Figure 6.2
Effect of initial imperfections on the 25m deep Frame C
Figure 6.3
Effect of the distribution of hydrodynamic loading on the response of the 25m deep Frame C

Figure 6.4
Effect of the distribution of hydrodynamic loading on the response of the 40m deep longitudinal Frame B
Figure 6.5
X-joint 16 nonlinear force-displacement relationship

Figure 6.6
X-joint 17 nonlinear force-displacement relationship
Figure 6.7
Effect of joint flexibility on the 25m deep Frame C

Figure 6.8
Axial force variation in springs at X-joints 16 and 17 of 25m deep Frame C
Figure 6.9
Effect of material behaviour on the response of the 25m deep Frame A

Figure 6.10
Effect of material behaviour on the response of the 25m deep Frame C
Figure 6.11
Effect of material behaviour on the response of the 80m deep longitudinal Frame B
7. DISCUSSION

7.1 SUMMARY

Ultimate strength analyses are being increasingly used to determine the reserve and residual strength, the redundancy and mode of failure of the jacket. While there is a variety of performance measures which may be used to assess the results of ultimate strength analyses, only the reserve strength ratio (RSR) is widely used by the industry. Indeed Table R.5.2b of API RP2A-LRFD (1997) which relates to assessment criteria of ultimate strength analyses of fixed platforms in non Gulf of Mexico US areas specifies a minimum reserve strength ratio (RSR) of 1.6 for structures with high consequences of failure and a minimum RSR of 0.8 for structures with low consequences of failure. Other performance measures are not considered in the assessment process. Furthermore, these magnitudes of RSR are intended for use in US waters and are not directly applicable to structures in the UK sector of the North Sea. One of the main objectives of this study is to draw attention to the various performance measures in existence and to establish which of these should be routinely addressed. These and others were reviewed in Chapter 2 of this document. A number of performance measures, which reflect the pre-ultimate and post-ultimate degree of redundancy in the system, were developed and tested for a range of structural frames. The performance measures were first validated for a range of frames representative of jackets in the UK Sector of the North Sea. Next, sensitivity studies were carried out to validate the performance measures against variations in geometric, material and loading properties representative of 'typical' offshore structures.

7.1.1 Results of base study

Fifty-eight two dimensional pushover analyses were carried out using both a plastic hinge and a distributed plasticity formulation. In all cases joint behaviour was assumed to be rigid and initial imperfections were ignored. These assumptions are typical of modern offshore structures where jackets are designed such that failure occurs in the members and the braces rather than the joints. In addition, the influence of initial imperfections is becoming less relevant to modern structures, where quality control is considerably improved.

The frame configurations are representative of the wide variety of offshore jackets in the UK Sector of the North Sea. The frames had water depths varying between 25m and 140m. Both two-legged and four-legged frames were considered. The number of bays varied, between two to five, as a function of water depth. X, K, inverted K and single diagonal bracing configurations were considered. The API ductility requirements were not met by all bracing configurations although all exist in UK sector structures.

Based on the background study reported in Chapter 2, the following three different measures of reserve strength were identified:

- reserve strength between first component damage and the ultimate strength of the jacket
- reserve strength between first component failure and the ultimate strength of the jacket
- degree of redundancy (and ductility) after the ultimate strength of the jacket has been reached.

To this end, the following performance measures were selected and tested for all the frame configurations:

- $R_1 =$ environmental load at ultimate / environmental load at first plastic hinge.
- $R_2 =$ environmental load at ultimate / environmental load at first component failure (RF)
- $R_3 =$ environmental load at twice the ultimate deflection / environmental load at ultimate (RS)
- $R_4 =$ energy at environmental ultimate load / energy at first member failure
- $R_5 =$ energy at twice the ultimate deflection / energy at environmental ultimate load.
- $R_6 =$ weight of jacket ($W_s$) (kN)
- $R_7 =$ fabrication cost (no)
- $R_8 =$ reserve strength ratio (RSR)

As discussed in Chapter 5, the reserve strength ratio ($R_s$) is usually defined as the ultimate load divided by the design load. The measure of reserve strength which will be adopted in this study is the ultimate environmental load divided by the environmental design load. The environmental design load, referred to above, is the load at which the most highly utilised component reaches its maximum allowable stress (in WSD terminology). The results obtained from a pushover analysis are usually in the form of a load factor versus displacement. The load factor is applied to the environmental design loads, to obtain an ultimate environmental load.

The first performance measure, $R_{ps}$, reflects the available margin of reserve strength between first component damage and the ultimate strength, while the second and fourth performance measures, $R_3$ and $R_4$, reflect the available quantity of reserve strength between first component failure and the ultimate strength. The two performance measures, $R_3$ and $R_4$, reflect the strength beyond the ultimate strength up to a point corresponding to twice the ultimate deflection. The next two performance measures $R_5$ and $R_6$, reflect the cost of the jacket in terms of steel weight ($R_s$) and cost of fabrication. Finally, the commonly used Reserve Strength Ratio (RSR or $R_s$) is also determined.

The above performance measures were used to assess the sensitivity of both ultimate strength and post-ultimate ductility and energy absorption capacity to framing configuration, member capacity and joint capacity. To provide a high degree of confidence in the results, both load-based and energy-based performance measures were used.

For the shallow 25m frames and to some extent the 40m frames, there is a small margin of reserve strength between first component damage (or first component failure) and the ultimate strength. This low margin of pre-ultimate redundancy was observed in all the different framing configurations corresponding to the 25m and the 40m frames. The results showed that the framing configuration has a very significant effect on the post-ultimate ductility and redundancy for the jacket depths, as can be seen in Table 7.1 which shows the variation of the available redundancy and ductility with the type of bracing for the 25m frames.

The highest degree of redundancy is achieved in X braced frames with horizontal component at plan framing levels, followed by X bracing without horizontals, single diagonal and X bracing. The above results confirm the applicability of the API ranking of framing configurations to ensure adequate ductility to steel jackets in the UK Sector of the North Sea. However the API guidelines seem to consider the framing configurations as the sole important parameter in determining the available degree of redundancy and ductility in steel jackets. The role of component strength, in particular elastic utilisation ratios, is not clearly recognised.
Table 7.1
Effect of framing configurations on ultimate strength and post-ultimate ductility of frames

<table>
<thead>
<tr>
<th>Type of bracing</th>
<th>Ranking of redundancy and ductility</th>
<th>$R_s$</th>
</tr>
</thead>
<tbody>
<tr>
<td>X with horizontals</td>
<td>1</td>
<td>3.0</td>
</tr>
<tr>
<td>X without horizontals</td>
<td>2</td>
<td>2.8</td>
</tr>
<tr>
<td>Single diagonal</td>
<td>3</td>
<td>2.5</td>
</tr>
<tr>
<td>K</td>
<td>4</td>
<td>1.9</td>
</tr>
</tbody>
</table>

$R_s$ = energy at twice the ultimate deflection / energy at environmental ultimate load

Jackets with very close reserve strength ratios were found to have significantly different pre and post ultimate measures of redundancy. As mentioned above, X bracing configurations led to the highest margins of redundancy and energy absorption capacity. The cost of fabrication, reflected in terms of the additional number of welds associated with the bracing configuration, was found to be highest for X bracing. However the weight of the jacket, another indication of cost, was not always higher for X bracing configurations.

It should be recognised that the different performance measures, developed and validated in this study, reflect different measures of reserve strength and should therefore be used together to obtain insight into the ultimate strength and post ultimate behaviour of offshore jackets.

7.1.2 Results of sensitivity study
Forty seven, two-dimensional pushover analyses were carried out to:

- Assess the sensitivity of the performance measures to member capacity, joint capacity and distribution of hydrodynamic loading, and
- Assess the ranking of the framing configurations, when variations in member capacity, joint capacity and distribution of hydrodynamic loading are accounted for.

To these ends the effects of the following parameters were studied:

- Initial imperfections
- Stress-strain steel characteristics
- Distribution of hydrodynamic loading
- Joint strength and flexibility.

Performance measures were found to be most sensitive to variations in joint strength. However the ranking of the bracing configurations, which was developed in the previous chapter for a range of frames and is shown in Table 7.1, remains valid when variations in member capacity, joint capacity and distribution of hydrodynamic loading are accounted for.

7.1.3 Two dimensional versus three dimensional studies
The conclusions of this study are based on two dimensional ultimate strength analyses and, therefore, their applicability to typical three dimensional jackets should be assessed. In particular, the effects of multiplanar actions on the ranking of the different bracing configurations should be addressed.
One of the few studies to examine the effects of multiplanar bracing on the reserve strength of offshore jackets is that carried out by BOMEL through the Joint Industry Tubular Frames Project (BOMEL, 1998). Both experimental and analytical investigations were carried out for Y, K, X and double X planar joints in isolation and within the multiplanar configuration. The results indicated that the out-of-plane bracing restrains the chord deformations (hence increasing capacity) and results in out-of-plane load transfer which influences the response.

However, the sensitivity of both the ultimate strength and post-ultimate redundancy to variations in the out-of-plane bracing configurations remains an issue yet to be extensively addressed.

7.2 RESERVE STRENGTH OF FRAMES

7.2.1 Alternative loadpaths and sources of reserve
Analytical results have demonstrated that X-braced panels offer an alternative load path to resist loads. The panel may therefore sustain increasing load even after a compression brace buckles, provided members are not too slender and the supports do not precipitate rapid unloading from the compression member. Similarly, diagonal bracing where members are inclined alternately (between compression and tension) provides an alternative tension load path to counter load-shedding from a buckling compression member. Conversely K bracing offers no alternative load paths through the panel once a member (or joint) fails.

Beyond panel failure, reliance is placed on the surrounding structure and the relative capacity of the legs and adjacent panels. Members which are lightly loaded under standard design conditions can also play an important role in redistributing loads, and their omission (eg. to save weight) can lead to progressive collapse in the event of extreme environmental load. A fractional increase in structural weight can preserve structural integrity and increase the global capacity.

Similarly assessment of simple 3D structures (Pike and Grenda, Soreide et al, Gebara et al) has demonstrated the role of plan bracing in transferring loads between frames so that the full structure can be utilised.

It must be emphasised that redistribution depends on various parameters including the relative strength and stiffness of the alternative load paths as well as the configuration.

Reserve strength within jacket structures is required to resist extreme loads which are not accounted for in accepted elastic design procedures. In assessing the safety of offshore installations, it is now recognised that the risks of damage from vessel impact and dropped objects or abnormal environmental loading (eg. hurricanes, typhoons, earthquakes) are not negligible. There is therefore a requirement to demonstrate that structures can resist these loads without catastrophic collapse. The magnitude of the loading determines that elastic resistance cannot reasonably be provided and plastic redistribution must be exploited. Through this work sources of reserve strength and the role of frame behaviour have been identified. In addition, the potential reduction in reserve strength due to traditional optimisation schemes has been highlighted.

For offshore structures the operator may be concerned to quantify the load the structure can sustain in excess of the design value. On that basis the following three separate measures of reserve strength have been identified (see Figure 7.1) and used to rank the various bracing configurations considered in this study:
- Reserve strength between first component damage and the ultimate strength
- Reserve strength between first component failure and the ultimate strength
- Redundancy and ductility beyond the ultimate strength and up to a point corresponding to twice the ultimate deflection.

The reserve strength ratio, defined as the ultimate environmental load divided by the design environmental load was also used to rank the various bracing configurations.

It can be seen from the above discussion that the degree of robustness is a function of the structure's redundancy, ductility and capacity as depicted in Figure 7.2. A structure is considered robust if it has high levels of ductility and redundancy, in addition to a large capacity. For a structure to be robust, it must satisfy ductility, redundancy and capacity requirements. For example, if the performance measures used in this study are used to quantify the above criteria then one possible performance measure for robustness may be expressed as:

\[
\text{Robustness measure } R_s = R_1 \times R_2 \times R_3 \times R_4 \times R_5
\]  
\text{Eqn (7.1)}

where

- \( R_1 \) = environmental load at ultimate / environmental load at first plastic hinge.
- \( R_2 \) = environmental load at ultimate / environmental load at first component failure (RF)
- \( R_3 \) = environmental load at twice the ultimate deflection / environmental load at ultimate (RS)
- \( R_4 \) = energy at environmental ultimate load / energy at first member failure
- \( R_5 \) = energy at twice the ultimate deflection / energy at environmental ultimate load.

\( R_s \) = reserve strength ratio (RSR).

Both \( R_1 \) and \( R_2 \) reflect the degree of redundancy before the ultimate load is reached. Therefore, only one was chosen. \( R_4 \) and \( R_5 \) were not included in the above equation, but they may be used in optimisation schemes aimed at minimising the total life cycle cost of an offshore structure. It can be seen that a low value for any of the five performance measures on the right hand side of Equation 7.1 would lead to a reduction in the robustness measure \( R_s \). Equation 7.1 should be used in conjunction with constraints on the performance measures \( R_1, R_2, R_3, R_4, \) and \( R_5 \) to ensure that none of the individual measures fall below their acceptable criteria. The setting of criteria for individual performance measures and the validation of the robustness performance measure are discussed further in the recommendations section (Section 9.2). By calculating the above performance measure for the structures considered in this study, the ranking obtained using the post ultimate redundancy \( (R_u) \) was confirmed. However, a more detailed study is required to relate robustness to the available air gap and the system probability of failure.

Robustness may be expressed in an alternative manner, as a function of the relationship between the structure's undamaged and damaged capacities (Figure 7.3). In this figure, damage (the horizontal axis) is defined as the number of components which have failed divided by the number of components required to fail in order for the structures to collapse. A structure where a low degree of damage will result in a large drop in capacity is considered not robust, while a structure that can withstand a large degree of damage before it suffers any reduction in capacity is considered as very robust.

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7.3 COMPARISON WITH EXPERIMENTAL RESULTS OF THE JOINT INDUSTRY TUBULAR FRAMES PROJECT

Four two-bay X-braced frames were tested to collapse with lateral load applied to the top of the frame under displacement control in Phase I of the JIP Frames Project (Billington et al, 1993; Bolt et al, 1994a and b). The bases of the frames were hinged. Frames I and III were designed for member failure and will therefore be considered in this section (Figure 7.4). The horizontal brace was omitted in the latter case to investigate the influence of the member with a low elastic utilisation factor on the collapse mechanism.

Frame I was designed with a thickened joint can at the top bay X joint (Figure 7.4) so that the compression diagonal was the critical component. The global response is shown in Figure 7.5 where it can be seen that the first component failure occurs in the lower compression brace of the top bay after the ultimate strength have been achieved. The load in the tension brace remains constant at yield, while the load in the damaged compression brace is reduced. However, the residual strength gradually increases from Scans 13 to 17.

First yield occurs between Scans 7 and 8, nevertheless the frame sustains increasing load, albeit at a reduced stiffness as load is carried by the alternative compression diagonal. This confirms the results obtained in this study regarding the available margin of reserve strength between first component failure and the ultimate strength.

Frame III was identical to Frame I, except that the mid-height horizontal was omitted (Figure 7.4). This member in Frame I carried negligible load in the elastic regime (Figure 7.5) and might be omitted in practice to reduce structural weight. The global response is shown in Figure 7.6. The top bay lower compression brace buckled just after the ultimate load was reached, shedding load via the alternative top bay (tension) diagonal directly into the bottom bay compression member, which became visibly bowed. The alternative load path provided by the X bracing ensured that the overall frame capacity was maintained with no rapid reduction in load. However, from Scan 8 the upper bay tension member began to yield and the bottom bay compression member buckled with increasing displacement at Scan 11.

The sequence of failures, without the mid-height horizontal to evenly distribute loads to the bottom bay reduced the residual frame capacity significantly (Figure 7.7). Again, this confirms the results obtained from this study regarding the effect of horizontal members on the post-ultimate ductility and redundancy in the system.

In Phase III of the JIP Frames Project, a series of three-dimensional tests were carried out to establish the effects of nonlinear joint/member behaviour on three dimensional frame behaviour and collapse mechanisms.

The main objectives of the Frames Project are to provide a rigorous quantified demonstration of the nonlinear response of 3D structures and to validate techniques and practices for predicting ultimate system behaviour.

Figure 7.8 shows the frame configuration and the three load cases considered in the study.

The test structure was carefully scaled to approximately one third the size of a southern North Sea jacket, with components proportioned to be representative of offshore practice. Characteristics of past and present structures were introduced to provide data for assessment and design. Both X and K braced configurations were examined and joints and members were loaded in different manners to achieve member and/or joint failure.
Motivated by the need to study the behaviour of structures under extreme loading scenarios, static overload tests were combined with cyclic scenarios examining the ability of a damaged structure to survive the extreme event and subsequent storms.

Revealing results and far reaching implications for the design and lifetime integrity management of offshore structures have emerged from the large scale structural tests. In addition to the physical insight provided by the tests, the data provide a valuable framework for refining and validating the capabilities of existing software. These findings and conclusions will be shortly be available to the participants in the Joint Industry Frames Project.

7.4 BACKGROUND TO CARRYING OUT SENSITIVITY STUDIES

Ultimate strength analyses have traditionally been used to assess the integrity of existing structures, and various structural configurations at the design stage, to accidental and extreme loading. In particular it is now recognised that it is uneconomical to design offshore structures to remain elastic under accidental and extreme loading conditions and, therefore, ultimate strength analyses are increasingly being used to assess the contribution of any inelastic reserve strength of the jacket to resisting extreme and accidental loading.

More recently, ultimate strength analyses are increasingly being used to assess the criticality of members, to validate and modify inspection schemes and to assess various repair schemes.

It may be argued that the conventional use of ultimate strength analyses relates to a specified set of loading criteria. However, even under such conditions, it is important to assess the effect of variations in component strength, and uncertainties in the model and the loadings, on the overall system strength. For the more recent uses of ultimate strength analyses, it becomes important to carry out sensitivity studies to assess the criticality of members, which in turn will be used to develop inspection schemes. To this end, the following parameters should be considered:

- Variations in component strength including yield strength and post-ultimate component behaviour (brittle, ductile and brittle-ductile).
- Variations in loading magnitude and distribution due to uncertainties in the data.
- Variations in the sequence of component failure and effect on the overall redundancy of the system.
- Variations in initial imperfections or presence of defects or damage and effects on the component buckling strength and overall system strength.
- Variations in joint flexibility and effects on both the failure mode and the reserve and residual strength of the jacket.
- Variations in the soil strength and effects on both the failure mode and the reserve and residual strength of the jacket.

The sensitivity studies carried out in this report demonstrate some of the effects of the above parameters. The sensitivity of the performance measures developed in this study to some of the above parameters was studied in Chapter 6. This may also serve as a guidance on how the effect of these parameters may be studied in the context of ultimate strength analyses.

For example, the effect of initial imperfections may be accounted for by introducing an initial out-of-straightness in the compression braces. Various stress-strain steel characteristics may
be modelled by varying the strain hardening parameter for a particular 'material type'. In addition, the effects of joint flexibility and strength may be accounted for by the use of nonlinear springs which have been developed from experimental data or more detailed finite element analyses.

A limited number of sensitivity studies have been carried out for two dimensional structures. The results from these studies indicate that variations in component strength may lead to significant variations in system strength. However, the more complex the structure, the higher its capacity for redistribution and the less likely that variations in the strength of an individual member will affect the system strength. Therefore, it is recommended that further work be carried out to investigate to what extent sensitivity studies should be carried out in three dimensional analysis.

Having established that certain applications of ultimate strength analysis require the use of sensitivity studies, a choice between deterministic and reliability-based methodologies should be made.

### 7.5 RELIABILITY BASED VERSUS DETERMINISTIC SENSITIVITY-DEPENDENT STUDIES

Both deterministic and reliability based sensitivity studies are being increasingly used to study the effects of variations in different structural and loading parameters on the reserve and residual strength and the overall redundancy in the system.

Probabilistic methods clearly involve repeated deterministic analysis. At this stage of development of hardware capability, nonlinear deterministic structural analysis software such as SAFJAC, which models member, joint and foundation nonlinear behaviour, has so far achieved limited use in probabilistic evaluations and to date the tendency has been to use simplified analytical approaches involving much less demand for computer time and storage in order to attain probability distribution for strength and other performance measures. BOMEL believes that with the continuing improvement in hardware capability, rigorous probabilistic analyses are now feasible. Furthermore, sensitivity studies in this report have demonstrated that several of the parameters which may be considered as variables in probabilistic analysis do have an important effect on performance. Therefore, it is recommended that probabilistic analysis are carefully evaluated to ensure that the full behaviour which will significantly affect performance is properly modelled.

While reliability studies offer a consistent methodology for assessing the variations in the component strength on the overall system strength, various deterministically-based scheme targeted at assessing system effects have emerged (e.g. Pandey and Barai, 1997).
Figure 7.1
Schematic diagram of a load deflection curve showing different areas of reserve strength

Figure 7.2
Robustness as a function of ductility, redundancy and capacity
Figure 7.3
Robustness as a function of structural damage

Figure 7.4
Two bay X-braced test frames
Figure 7.5
Frame I global and local member responses

Figure 7.6
Frame III global response
Figure 7.7
Comparison of Frames I and III responses

Figure 7.8
Original BOMEL 3D frames configuration and loading scenarios
8. CONCLUSIONS AND RECOMMENDATIONS

The ability to predict the reserve strength of jacket structures is now of considerable importance to the offshore industry. There is a requirement to extend platform operating life despite more onerous loadings and more stringent code requirements than at the design stage. Furthermore, risks of extreme events which cannot be resisted elastically, have been identified and adequate system reserve is therefore a necessary requirement in configuring new jackets. More recently, ultimate strength analyses are also being used to determine critical components whose integrity is essential for maintaining the system reserve strength. Critical components are then identified to receive a higher degree of quality assurance, inspection and maintenance schemes. In the same manner, the inspection of uncritical components may be postponed or eliminated.

The recognition of the importance of reserve strength technology has been met by the development or adaptation of a range of nonlinear software to perform collapse analysis of jacket structures. These embody different approximations and numerical devices with a view to ensuring that the complex nonlinear problems can be analysed efficiently and to sufficient accuracy. In addition to calibration at the component level, benchmarking against available test data is increasingly being carried out.

Notwithstanding recent advances in software capabilities, various sources of physical, statistical and modelling uncertainties are yet to be resolved. These include uncertainties in loading data, foundation behaviour, joint behaviour and variations in component strength. The presence of these uncertainties, coupled with the more novel applications of ultimate strength analyses, indicates that it may be appropriate to carry out an extensive set of analyses rather than a single analysis corresponding to a unique set of loading data, foundation and joint strength and component material properties. Such sensitivity studies can be carried out either in a probabilistic or in a deterministic manner.

This study has reviewed the various probabilistic performance measures and outlined a methodology for carrying out deterministic sensitivity-based studies. However, both of the above are relatively recent and many of the issues raised remain unresolved. The principal recommendation of this report, is therefore, that procedures for validating the accuracy of the structural model and corresponding analysis results should be developed. These procedures should take into account the different applications of ultimate strength analyses.

This study has focused on reserve strength and recent lessons from reliability-based and deterministic sensitivity-dependent ultimate strength analyses. Specific conclusions from the present study are given in Section 8.1, while recommendations are outlined in Section 8.2.

8.1 CONCLUSIONS

Base study

- Bracing configurations play a very important role in determining the reserve strength of jacket structures. For example, the absence of horizontal members between the legs may lead to a significant drop in the post ultimate strength if there are no alternative load paths available (eg. through stiff plan bracing).
- The API guidelines on bracing configurations in seismically active areas, may be applied to assess the level of redundancy and alternative load paths of offshore structures in the UK sector of the North Sea.
• The above API guidelines are limited to two-dimensional vertical frame configurations and should be extended to account for the effects of bracing configurations at horizontal plan framing levels.

• While the API guidelines recognise the effects of framing configurations on the redundancy and ductility of offshore jackets, other important parameters such as component elastic utilisation factors are not addressed.

• Jackets with identical ultimate strengths may exhibit significantly different post-ultimate behaviour and, therefore, for a single analysis, it is necessary to examine the values of different performance measures.

• Performance measures for determining the pre-ultimate and post-ultimate degree of redundancy in offshore jackets have been developed and used to rank various bracing configurations.

• The reserve strength from the alternative load paths through X-braced panelling is demonstrated in contrast with the lack of redundancy in K-bracing or single diagonal bracing.

• The work carried out in this study provides good insight into the effects of framing configurations on the ultimate and post-ultimate performance measures of two-dimensional jackets. Three-dimensional effects, which are very important particularly when there is a lack of redundancy in two-dimensional frames have not been considered. While the work carried out in this study is very useful for framing arrangements in two-dimensions, three-dimensional out-of-plane bracing arrangements should also be examined. For example, it is possible to achieve load redistribution in a three-dimensional jacket even after a two-dimensional vertical frame fails. Therefore, it is recommended that further studies are required to study three-dimensional effects in more detail.

• As a result of this study, various variables which reflect different measures of structural performance have been identified (eg cost of fabrication, weight, reserve strength, energy absorption capacity, etc). Each of these variables is relevant and provides additional insight into system performance and therefore all can contribute to optimising performance. Clearly some form of optimisation scheme aimed at balancing all the above variables is required. It is recommended that the application of formalised optimisation theories to this topic is investigated.

Sensitivity study
Pushover analyses are often carried out with certain parameters such as loading conditions and material properties, held constant. A limited number of parameters were chosen to demonstrate the use of sensitivity studies in deterministic ultimate strength analyses. The objective of the sensitivity study is to explore if, and when, the effects of variation in certain loading and structural parameters may be neglected. To this end, the following four parameters were considered:

• Strain hardening parameter
• Initial imperfections
• Proportion of still water to hydrodynamic loading
• Joint flexibility and strength.

Results from pushover analyses demonstrate that variations in any of the above parameters may significantly alter the ultimate and post-ultimate response of fixed steel jackets.
Conclusions regarding the particular cases which were considered are presented below:

- Post-ultimate component material behaviour was examined by varying the strain hardening factor. It was concluded that as the strain hardening parameter is increased, the residual strength of the jacket increases.

- Joint flexibility and strength play an important part in determining the ultimate and post ultimate behaviour of structures. Ignoring joint flexibility effects in cases where joints are highly utilised, may lead to unconservative results.

- While initial imperfections affect the compressive strength of individual components, their effect on the system reserve strength was found to be negligible for the cases considered.

- The ratio of still water to environmental loading was varied to study the effects of the distribution of the hydrodynamic loading on the reserve strength of jackets. For the cases considered, the failure mode and, therefore, the reserve strength, were not affected by variations in the distribution of the hydrodynamic loading. This project has therefore provided the basis for reliable determination of performance measures for pushover analysis of offshore jackets, and has led to the introduction of ranking scenarios.

The extension of the sensitivity study to examine the effects of the above parameters in a more detailed manner and to account for variations in additional parameters is discussed in Section 8.2, Recommendations.

### 8.2 Recommendations

The work carried out has led to a considerably improved understanding of the effect of framing configurations on the ultimate and post-ultimate response of jackets. Performance measures and systematic ranking techniques have been developed and are significantly in advance of those previously available. Notwithstanding the advances made in this study, the following areas should be developed further to provide a clear methodology for assessing and optimising structural system performance:

1. The effects of out-of-plane bracing should be examined by carrying out a series of pushover analyses for a variety of existing three dimensional platforms. The main objectives (and corresponding deliverables) of such a study are to study the effect of out-of-plane bracing configurations on the ultimate and post-ultimate behaviour of 3D fixed offshore structures and to confirm the validity of the performance measures developed in this study when applied to assess the behaviour of 3D fixed offshore structures. As a first step, a variety of 3D fixed jacket structures representative of existing offshore platforms in the UK sector of the North Sea will be selected. Next a design premise, similar to the one adopted in this study will be developed. Finally pushover analysis will be carried out and the performance measures developed in this study will be calculated to compare behaviour.

2. This study has identified various measures which may be used to reach recommendations regarding structural performance. Each provides important insight into structural performance and together they provide a comprehensive understanding. It is BOMEL's view that the application of optimisation techniques to this group of measures would result in improvements in robustness, structural reliability and safety and therefore should be pursued as a next step.
3. Based on the two dimensional structures used in this study, reliability-based sensitivity analyses may be carried out to examine the sensitivity of the performance measures identified in this study to uncertainties in a number of input parameters (e.g. magnitude of imperfection and yield stress). The objectives (and corresponding deliverables) of such a study are:

i) to determine probabilistic distribution of the performance measures developed in this study

ii) to rank the effects of uncertainties in the various input parameters on the uncertainty of each of the performance measures.

By using the three dimensional structures described in Point (1) above, it may be possible to extend the sensitivity study to three dimensional structures.

4. The optimisation study, discussed in Point (2) above, may be extended to the probabilistic domain by using the probabilistic distributions of the performance measures determined in Point (3) above. In this manner an optimisation scheme which is subjected to probabilistic constraint equations could be developed with the aim of improving safety and reliability.
9. REFERENCES


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