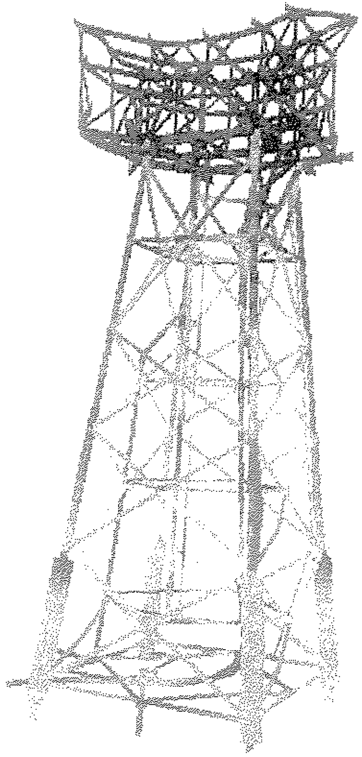
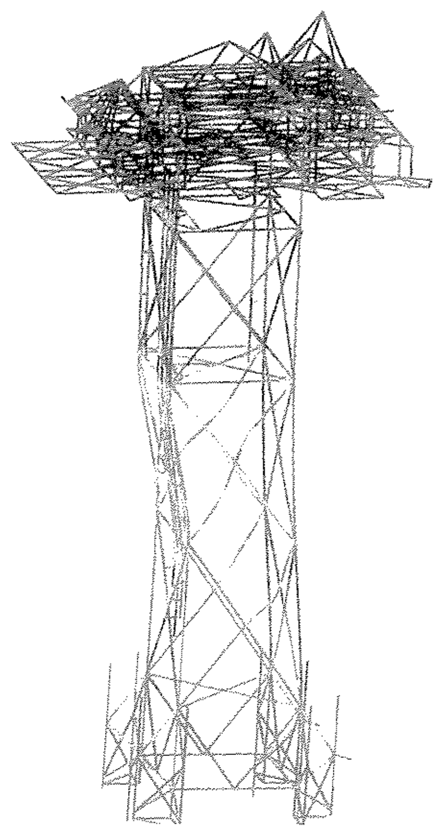
292 A6

ULTIGUIDE

BEST PRACTICE GUIDELINES FOR USE OF NON-LINEAR ANALYSIS METHODS IN DOCUMENTATION OF ULTIMATE LIMIT STATES FOR JACKET TYPE OFFSHORE STRUCTURES





PREPARED BY

DNV - SINTEF - BOMEL

APRIL 1999

Foreword

Non-linear analyses help the engineer to a better understanding of the structural behaviour when the structure approaches its load bearing limits. This guide is intended to provide the engineer guidance to undertake analyses and to interpret results from such analyses

The Ultiguide document is produced by the Joint Industry Project named Ultiguide Phase 2. The sponsors of this project are:

Aker Maritime AOP
Amoco Corporation
Brown & Root Ltd.
Det Norske Veritas A.S.
Exxon Production Research Company
Health and Safety Executive
Minerals Management Service
Norsk Hydro
Phillips Petroleum Co. Norway
Shell UK Exploration
Statoil

The project is carried out as a joint effort by Det Norske Veritas, SINTEF, and BOMEL with contributions from Aker Maritime AOP and Brown & Root Ltd.

Please note that whilst every effort has been made to ensure the accuracy of the Ultiguide neither the members of the project team nor the sponsoring companies or organisations will assume any liability for any use thereof.

Also note that sponsorship of the project does not imply full endorsement of every element of the Ultiguide document.

The Ultiguide document is published by Det Norske Veritas, Veritasveien 1 N-1322 Høvik, Norway.
Website http:/www.dnv.com

The Ultiguide document is prepared by the following project team: Gunnar Solland Det Norske Veritas (Project manager and editor), Øyvind Hellan SINTEF, Helen Bolt BOMEL; Irene Fuglum Aker Offshore Partner and Sirous Yasseri Brown & Root.

The various chapters are written by:

INTRODUCTION
Gunnar Solland

ANALYSIS METHODS
Øyvind Hellan, Helen Bolt

STRUCTURAL BEHAVIOUR AND FAILURE CRITERIA
4.1, 4.2, 4.4 and 4.5 Øyvind Hellan
4.3 Helen Bolt

STRUCTURAL MODELLING
5.1, 5.2, 5.3 and 5.5 Øyvind Hellan
5.4 Helen Bolt

LOAD MODELLING
Øyvind Hellan

SOFTWARE REQUIREMENTS
Øyvind Hellan

EXECUTION OF ULTIMATE STRENGTH ANALYSES
Øyvind Hellan, Helen Bolt, Gunnar Solland,

STRUCTURAL RELIABILITY ISSUES
Gunnar Solland

REPORTING
Irene Fuglum

COMMENTARY
Øyvind Helland, Helen Bolt, Gunnar Solland

EXAMPLES
Irene Fuglum, Sirous Yasseri

The Ultiguide project was conducted by a Steering Committee with the following members:

Irene Fuglum	Aker Offshore Partner
John K.Smith, (Richard Beck, Joseph Gebara)	Amoco Corporation
Sirous F Yasseri,	Brown & Root
Inge Lotsberg,	Det Norske Veritas
Carl Brinkmann,	Exxon Production Research Company
Conrad DeSouza,	Health and Safety Executive
Stein Olav Drange,	Norsk Hydro
Charles E. Smith,	Minerals Management Services
Peng Zheng,	Phillips Petroleum Co. Norway
N.W. Snedden, (Harry van Langen)	Shell UK Exploration and Production
Lars G. Bjørheim,	Statoil

Content for Best Practice Guidelines

1 Introduction

The intent of this document is to provide guidance to an engineer engaged in non-linear frame analyses of offshore structures. The guide is written for engineers familiar with analysis of offshore structures.

Non-linear frame analysis provides a better understanding of the overall structural system compared with traditional design practice, which typically focuses on individual components with load effects determined from a linear frame analysis. This document provides guidance to an engineer to select parameters and interpret results from non-linear analysis.

At present, such analyses are being used most frequently in relation to the reassessment of existing structures. Consequently, the topics dealt with in this guide address the questions, which arise in connection with that type of work, but the recommendations will be valid also in other design situations.

The guide focuses on jacket structures as these are the most common type of existing offshore platforms. Nevertheless, the recommendations will also be valid to some extent for other types of structures. The guideline does not specify a recommended safety level, but is intended to be used in conjunction with existing codes and specifications.

This guideline provides guidance for the behaviour and the resistance of the components of a structure as well as of the structural system.

Elastic linear analyses possess the advantage that combination of loads can be made according to the principle of superposition. Furthermore, the analyst does not need to check that the actual cross sections are stable where plastic hinges form and that repeated yielding does not occur. Guidance on how these issues should be dealt with are presented in this document.

The document is structured in the following way:

Section 3 gives a brief overview of the analysis methods available to the engineer.

Section 4 describes the physical behaviour of the typical components in fixed offshore, i.e. *"Which effects have to be captured in a non-linear analysis?"*

Section 5 describes the FE modelling required to properly represent this physical behaviour in the non-linear analysis, i.e. the *"How to...?"* part of the problem.

Load modelling is treated in Section 6, while Section 9 sets up some requirements to the software used for non-linear pushover analyses.

Execution of the actual analysis is discussed in Section 7.

Section 8 discusses use of the pushover results in a safety / reliability context, and Section 10 sets up some requirements of the reporting of pushover results.

2 Definitions and Symbols

2.1 Definitions

The following definitions are used in this document.

beam-column frame component which in the general case is subjected to both axial compression and bending moments.

beam-column element finite element type used to represent members under combined axial and bending loading. The element type can represent elastic or inelastic bending behaviour and elastic or inelastic buckling behaviour of structural members. The element type includes interaction between axial force and bending moments; presence of bending moments reduces the axial capacity and influences the buckling behaviour, and presence of axial forces reduces the bending capacity and influences the inelastic bending behaviour. The element type also accounts for redistribution of internal forces (from axial to bending action if the member buckles, and from bending to membrane tension if the member fails in bending).

beam element finite element type used to represent bending members. The element type can represent the elastic or inelastic bending behaviour of structural members, with or without axial forces. However, the element is not able to represent buckling behaviour - the presence of axial forces only reduces the bending capacity of the element.

characteristic action the value of an action which has a prescribed probability of being exceeded within a reference period.

characteristic resistance the value corresponding to a specified fractile of the statistical distribution of the resistance.

column element finite element type used to represent axially loaded members. The element can represent buckling behaviour, with and without the presence of bending moments. However, the element is not able to represent beam bending. The presence of bending moments only reduces the axial capacity of the element.

cyclic action an action which is repetitive

design action the value obtained by multiplying the characteristic action with the partial factor.

design resistance the value obtained by dividing the characteristic resistance with the material factor or by multiplying with the resistance factor.

ductility the ability to deform beyond the proportionality limit without significant reduction in the capacity due to fracture or local buckling.

ductility limit the deformation level at which the component experiences a reduction in capacity below that which is compatible with its load-carrying function.

dynamic action action which may cause significant acceleration of the structure or structural elements.

environ-mental load multiplier (ELM) parameter that is used to increment the characteristic environmental actions until the structure has reached its ultimate capacity.

general beam-column models finite element type where the member stiffness and (in-) elastic behaviour is determined by numerical integration of the stress distribution at points across the section. These elements do not generally incorporate geometric non-linearity at the element level and multiple beam elements along a member are required to accurately model buckling responses.

reserve strength ratio (RSR) the ratio of a platforms ultimate lateral load carrying capacity to its 100-year environmental condition lateral loading, computed using present API Recommended Practice 2A procedures.

residual strength the capacity of a damaged structure.

phenomeno-logical models a modelling approach where component responses are prescribed a-priori by force-deformation relationships which can be empirically related to element geometry or determined through analysis. A single (often one degree-of-freedom) element represents the member behaviour.

plastic hinge beam-column models finite element type where the inelastic element behaviour (buckling or yielding) is modelled by plastic hinges (concentrated or distributed) governed by plastic flow theory (flow rule, normality) and a yield surface defined in terms of axial force and bending moments.

special purpose frame element a term which is used to describe a newly developed finite element type constructed by combining a beam element and a strut element as defined below. The element switches from (non-linear) beam behaviour to strut behaviour once a design capacity equation ("unity check") is satisfied. Post-buckling behaviour is represented by a strut element, i.e. bending moments are neglected in the post-buckling range.

strut element terminology often used to describe a special type of column element (Marshall Strut) where the buckling behaviour is described by one degree-of-freedom, following a prescribed P-delta curve. The element is not able to represent beam bending The presence of bending moments must be accounted for by reducing the axial capacity of the element.

ultimate limit state a state associated with collapse, or other similar forms of failure.

2.2 Symbols

The symbols used in the Ultiguide are listed below:

A	area
A_C	cracked area
C_m	moment factor
D	chord diameter
E	Youngs modulus, environmental load
F_{AR}	reduction factor for cracked joints
K	buckling length factor
L	length of tubular between ringstiffeners, live load
M	moment
M_{hU}	bending moment capacity reduced by hoop stress
M_p	plastic moment capacity
M_{Ux}	ultimate moment capacity about x-axis
N	axial force
N	axial force in member
N_{hU}	axial capacity reduced by hoop stress
N_p	plastic axial capacity
N_q	capped-end axial force
N_{Ux}	axial plastic yielding capacity
P	Permanent load
P_{Cr}	column buckling resistance
P_{Ek}	Euler buckling load
R	resistance
f_h	hoop stress
f_{he}	elastic hoop buckling stress
f_{hU}	ultimate hoop stress
f_y	yield strength
g	tubular joint gap
p	external hydrostatic pressure
q	distributed load
l	member length
r	radius of gyration
t	thickness
z	depth
z_R	depth of reduced resistance zone
α_C	slenderness parameter for local buckling of pipes
β	diameter ratio of brace to chord
δ	deformation
ε_{Cr}	critical strain
ϕ	resistance factor
γ	ratio of chord radius over chord thickness, unit weight of soil, safety factor
λ	slenderness
$\bar{\lambda}_K$	reduced slenderness
ζ	ratio of gap length over chord diameter

3 Analysis Methods

3.1 Non-linear methods in ultimate strength analysis

Conventional structural analysis practice (Figure 3-1) relies on idealised linear-elastic models to determine the internal forces in components of a structure. Their adequacy is then determined by comparing the applied element forces with parametric code-check capacity formulae that are based on isolated component failure data.

In ultimate strength analysis, (Figure 3-2) non-linearities associated with the plasticity and large deformations of components under extreme load are included explicitly in the element modelling. The analysis tracks the interaction between components as member end restraints are modified and internal forces are redistributed in response to local stiffness changes. The sequence of non-linear events leading to a global collapse mechanism and the associated system capacity are determined.

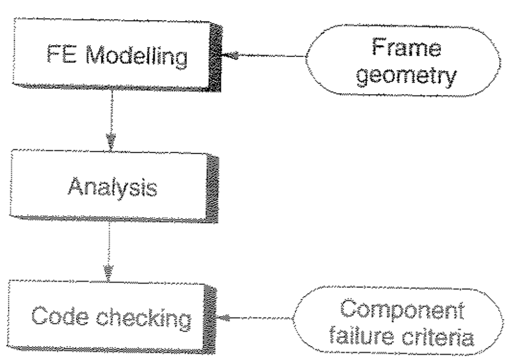


Figure 3-1 Conventional analysis design procedure

Thus, while the typical linear design process checks for the adequacy of each individual component, the non-linear ultimate strength analysis models the performance of the system as a whole.

3.2 Description of non-linear analysis methods

3.2.1 General

Four basic types of non-linear analysis techniques are available:

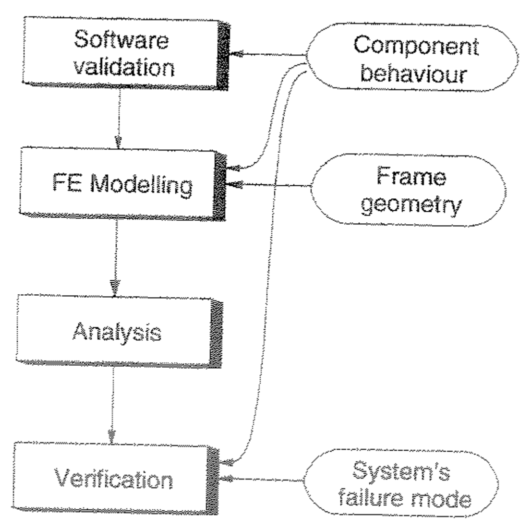


Figure 3-2 Non-linear ultimate strength analysis

- General purpose non-linear beam-column models
- Plastic hinge beam-column models
- Phenomenological models.
- Shell FE models

Different variations exist within each category, as discussed e.g. in Section 2.1 and Section 5.3.

The methods are semi-empirical, to varying degrees, having a theoretical basis but with some calibration to experimental data.

Features of the principal modelling approaches may be summarised although specific understanding of the chosen software must be gained by the user from manuals and test case calibrations.

3.2.2 General purpose non-linear beam-column models

Beam elements do not generally incorporate geometric non-linearity at the element level and multiple beam elements along a member are required to accurately model buckling responses.

The equations of equilibrium are usually evaluated in the deformed condition (large displacement solutions). The stiffness is integrated numerically from the stress distribution at points across the section due to the combined action of axial forces and moments.

Detailed stress-strain material modelling formulations including strain hardening may be available. These may require definition in a 'true' rather than 'engineering' format.

Components may also be modelled by shell elements, or with a combination of shell element (e.g. for joints, damage representation of members) and beam elements, albeit with a greater modelling and analysis demand.

3.2.3 Plastic hinge beam-column models

Certain techniques have been developed specifically for the ultimate strength analysis of space frame structures. The elements have been derived to model beam column behaviour and each member requires only a single element to model the buckling response.

Plasticity may be introduced by modelling the propagation of yield through the section and along the element length, or by the formation of idealised hinges. Plastic hinges are defined using the basic relationships of plasticity theory (flow rule, normality) and a yield surface (in terms of axial force and bending moments). Depending on the element formulation, hinges may form at the calculated locations of first fibre yield or at representative end and mid point locations.

Idealised elastic-plastic behaviour or gradual strain hardening may be accommodated related to the energy dissipated in the hinge.

The element specification can be adjusted to account for inherent imperfections or to calibrate responses to characteristic rather than mean component strengths given by test data.

3.2.4 Phenomenological models

In a phenomenological model, component responses are prescribed with force-deformation relationships which can be empirically related to element geometry or determined through analysis. A single element represents the member behaviour.

In some software, the type of failure must be anticipated for each component and loading mode prior to analysis and the element type and its non-linear characteristics defined accordingly, e.g.:

- Elastic members
- Buckling members

- Beam members
- Frame members
- Joints

Material non-linearity and initial imperfections are embodied in the phenomenological representation of test data. Section yielding may be determined from the full-section forces and a specified interaction surface.

3.2.5 Shell FE models

Components may also be modelled by shell elements, or with a combination of shell element (e.g. for joints, damage representation of members) and beam elements, albeit with a greater modelling and analysis demand.

3.3 Software validation

Whereas in conventional analysis practice, loading cases and component code check procedures are 'standardised', non-linear analysis programs differ in their treatment of component responses (Section 3.2) and the engineer must therefore understand the basis of the chosen representation and ensure it is appropriate. Chapter 4 describes the various component responses which non-linear analysis methods must represent. Chapter 5 details the modelling approach depending on the analytical technique adopted. Particular issues to consider include:

- Representation of relevant failure modes-user selections:
  - component types
  - capacity
  - deformation
  - load shedding
- Material modelling (yield) characteristic
- Treatment of inherent imperfections (residual stresses, fabrication / geometric tolerances).

Chapter 9 provides further guidance on software requirements and the essential steps for ensuring that the analysis tool and the manner of its use will deliver reliable results in the particular application.

3.4 Load application

The method of load application also depends on the software. In some cases distributed environmental loads can be applied, in others equivalent nodal loads must be evaluated. If neglected, care should be taken that local element loads would not significantly influence the response.

The principle of load superposition is not valid for non-linear analysis; the sequence and location of non-linear events is dependent on the pattern of load application. Appropriate loading strategies for offshore jacket structures are presented in Chapter 6.

4 Structural Behaviour and Failure Criteria

4.1 General

It is important that the non-linear analysis tool can predict failure in accordance with recognised failure criteria and design formulations.

The analysis tool and / or the modelling adopted should represent the non-linear behaviour of the structural elements that contribute to the failure mechanism with sufficient accuracy.

In a linear analysis, each component is sufficiently described by its stiffness and capacity. In a non-linear analysis it is also necessary to describe how each component interacts with the surrounding structure, i.e. to describe how the component fails, how it sheds loads onto the surrounding structure and how much deformation it can take before total severance.

Thus, stiffness, capacity, yield characteristics, post ultimate behaviour and ductility limits (if applicable) should be represented, including local failure modes such as local denting, local buckling, joint overload, joint fracture etc.

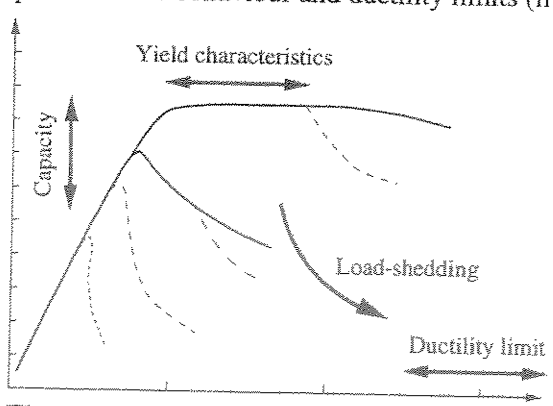


Figure 4-1 Description of non-linear component characteristics

In the following, the term "ductility limits" may refer either to the physical tearing capacity of the material, or, more often, to a limit of validity for the mathematical models used to describe the behaviour of the component.

The present section gives a brief description of the structural behaviour of components in framed offshore structures, along with examples of failure criteria defined for each behaviour mode. The focus is on the physical phenomena, i.e. *"Which effects has to be captured in a non-linear analysis?"*.

Section 5 describes the FE modelling required to properly represent these behaviour modes in the non-linear analysis, i.e. the *"How to...?"* part of the problem.

4.2 Members

4.2.1 General

There are two fundamental behaviour modes for individual members:

1. Steel yielding within the member cross-section due to a combination of axial and bending forces
2. Stability failure (column buckling) either elastically or in-elastically

Member behaviour in either of these modes will be influenced not only by the overall member dimensions (*D, t, L*), but also by local properties. These local properties require consideration of local buckling, denting and hydrostatic pressure effects. The behaviour may be further constrained by consideration of ductility limits and cyclic degradation.

4.2.2 Yielding / yield hinges

First fibre yield of a single member is of little concern as a global failure criterion for ultimate strength analyses. Further loading will lead to gradual spreading of the plastic region through the thickness of the section and along the member length, until the entire section is yielding.

Tests on steel members in bending and/or compression show that inelastic deformations tend to concentrate in regions with limited extension along the length of the member. These regions are denoted yield hinges or plastic hinges.

Stresses within a yield hinge can be re-distributed between bending action and axial loading, e.g. due to membrane action or frame action which constrain the deflection at member ends. The combined axial and bending stresses are limited by a yield surface as indicated in Figure 4-2 and Figure 4-5.

For a simply supported beam in bending, formation of a yield hinge will mark the limit of the load-carrying capacity. If the beam is restrained axially, further loads can be carried by activation of membrane effects in the beam, ref. Figure 4-2. Thus, formation of a yield hinge is not necessarily the limit of the load-carrying capacity; generally a structure or component will only fail once sufficient number of plastic hinges has formed to make a kinematic mechanism.

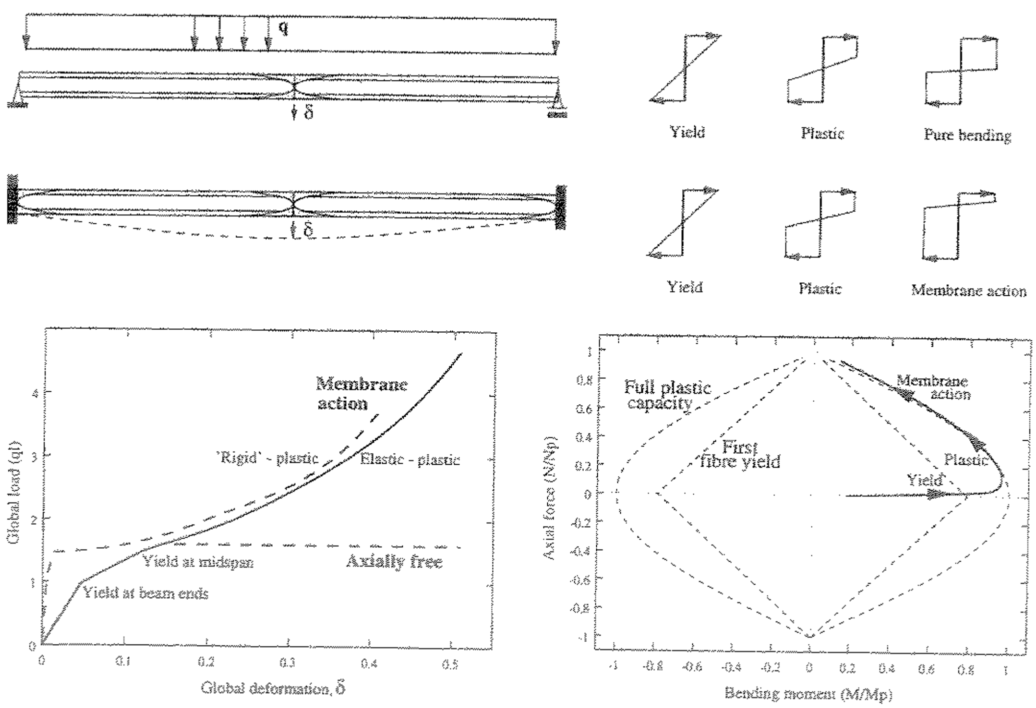


Figure 4-2 Yield hinges, beam bending and membrane effects

4.2.3 Column buckling

This is defined by loss of stability of a structural member due to axial forces. In conventional design practice, the buckling capacity is described by column curves such as shown in Figure 4-3 and Equation (4.1)

$$\frac{P_{Cr}}{N_{Ux}} = \begin{cases} 1-0.28\bar{\lambda}_k^2 & ,\bar{\lambda}_k \le 1.34 \\ 0.9/\bar{\lambda}_k^2 & ,\bar{\lambda}_k > 1.34 \end{cases} \qquad (4.1)$$

where

$$\bar{\lambda}_k = \frac{K\ell/r}{\pi\sqrt{E/f_y}} = \sqrt{\frac{N_{Ux}}{P_{Ek}}}$$

Slender members fail through elastic buckling (region 3 in Figure 4-3) and stocky members fail through plastic yielding (region 1). Most axial members in offshore structures fall in the intermediate region (region 2), in which failure is caused by inelastic buckling with inelastic deformations concentrated at specific locations (yield hinges / plastic hinges) along the column length.

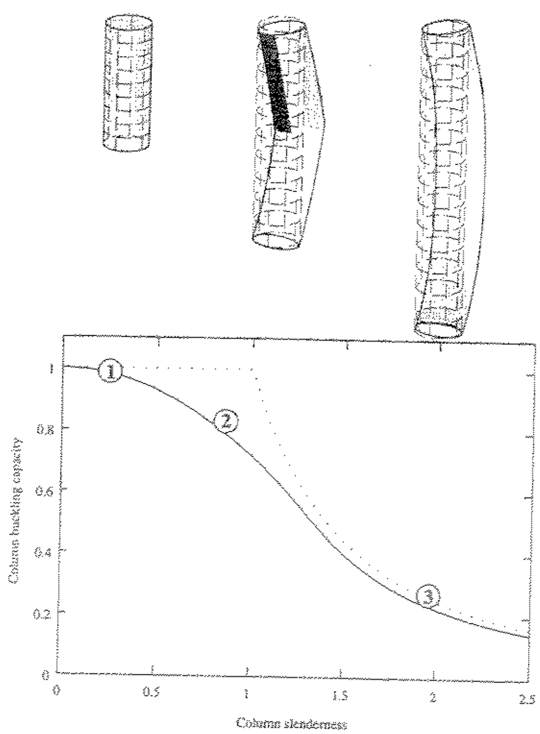


Figure 4-3 Column buckling

For members loaded by axial force and bending moments conventional design codes define failure when stability equations such as (4.2) become unity.

$$\frac{N}{P_{Cr}} + \frac{C_m M}{(1-N/P_{Ek})\,M_{Ux}} \le 1 \qquad (4.2)$$

Effects of cross-section geometry, boundary conditions, loading etc. are included through the C_m factor, the effective length factor K and choice of the appropriate column curve.

The moment amplification factor $\frac{C_m}{1-N/P_{Ek}}$ in the second term of Equation (4.2) is an approximate measure to include the second-order bending moment in the stability equation. See Figure 4-4.

In non-linear pushover analyses, buckling is not introduced as result of a separate event or 'check' in the analysis. The element stiffness is a function of the axial force in the element, and is continuously decreasing as the axial force in the member increases. At some axial force level the element stiffness is reduced to the extent that the column can no longer support additional loading. From this stage in the analysis, the axial force in the member has to be reduced in order to maintain equilibrium. Thus, column buckling is simply

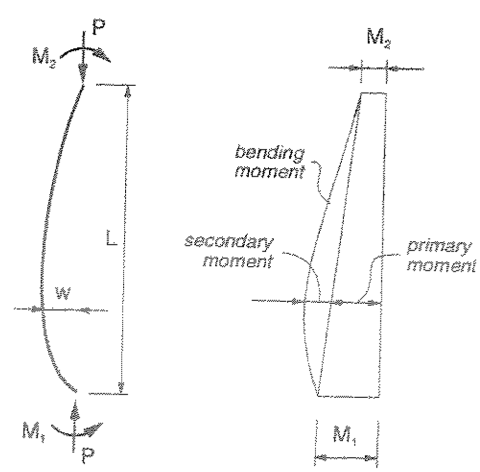


Figure 4-4 Second-order bending moment

defined by the peak force in the member P-δ behaviour, as indicated in Figure 4-5.

The non-linear formulations automatically calculate the total (first order + second order) bending moments, and also include the effects of cross-section geometry, boundary conditions, loading etc.

4.2.4 Local buckling

This is defined as loss of cross-sectional shape due to buckling of part of the cross-section exposed to compressive stresses (from axial loading or bending).

For tubular members, the cross-sectional capacity and the deformation capability (ductility) are function of the sectional slenderness, defined in terms of the D/t-ratio or non-dimensional section slenderness parameters such as
$\alpha_C = D/t \cdot f_y/E$.

In conventional design practice, the local buckling capacities are described by capacity curves such as the ones shown in Figure 4-7 and Figure 4-8 and Equations (4.3) and (4.4) /4/.

$$\frac{N_{Us}}{N_P} = \begin{cases} 1 & \alpha_c \le 0.102 \\ 1.047 - 0.457\alpha_c & 0.102 < \alpha_c \le 1.147 \\ 0.6 \;/\; \alpha_c & 1.147 < \alpha_c \end{cases} \tag{4.3}$$

$$\frac{M_U}{M_P} = \begin{cases} 1 & \alpha_c \le 0.0517 \\ 1.13 - 2.58\alpha_c & 0.0517 < \alpha_c \le 0.1034 \\ 0.94 - 0.76\alpha_c & 0.1034 < \alpha_c \quad 120 f_y/E \end{cases} \tag{4.4}$$

The onset of local buckling may be predicted from strain criteria or criteria formulated in terms of plastic work. The following formula is an example, originally derived for pipelines /18/ but found to give good correspondence with tests on tubular columns /6/.

$$\varepsilon_{Cr} = 8.5\left(\frac{t}{D}\right)^2 + 0.0021 \tag{4.5}$$

In addition to the capacity reduction, local buckling reduces the post/peak 'ductility' of a member significantly. Figure 4-6 shows P-δ characteristics for different cross section classes. The rapid load-shedding at section classes 3 and 4 (high D/t-ratios) should be noted.

In non-linear F.E. formulations, local buckling may or may not be included in the

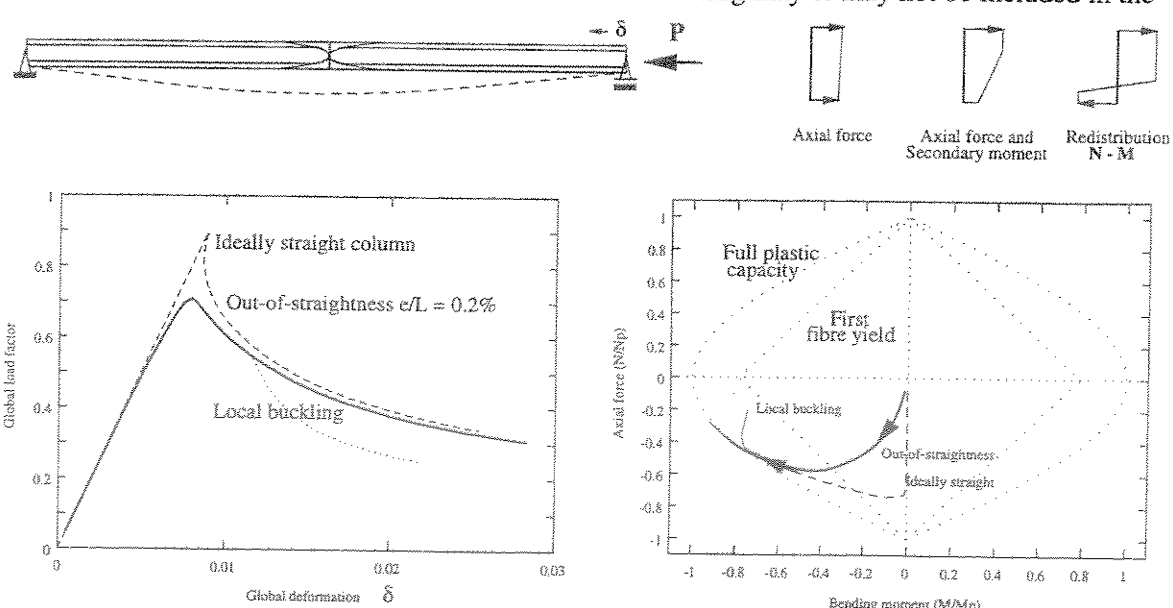


Figure 4-5 Column buckling in non-linear F.E. formulations

formulations. Some formulations include local buckling as a reduction in section capacity to the level prescribed by the design equations. Others have sophisticated models that account for local buckling in the post-collapse range, and growth of the buckle as member deforms. General shell F.E. formulations may be able to capture local buckling through the shell modelling, but initial geometric imperfections are generally needed to initiate the local buckle.

Table 4-1 Cross section classification

Cross section class	Classifi-cation	Performance
1	Plastic cross sections	The cross-section will attain its full plastic capacity. The section has sufficient ductility to maintain its full capacity at large inelastic deformations.
2	Compact cross sections	The cross-section will attain its full plastic capacity. Local buckling will take place in the post-collapse region and lead to reduced post-collapse capacity.
3	Semi-compact cross sections	The cross-section will attain its first fibre yield capacity. Local buckling will take place before the full plastic capacity is reached. Rapid load-shedding in the post-collapse region.
4	Slender cross sections	Elastic shell buckling

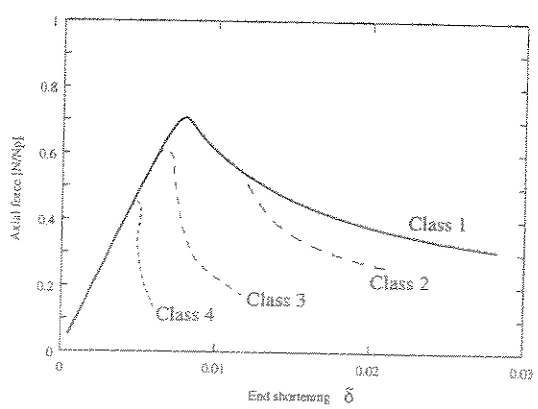


Figure 4-6 Typical P-δ curves for different section classes (D/t-ratios)

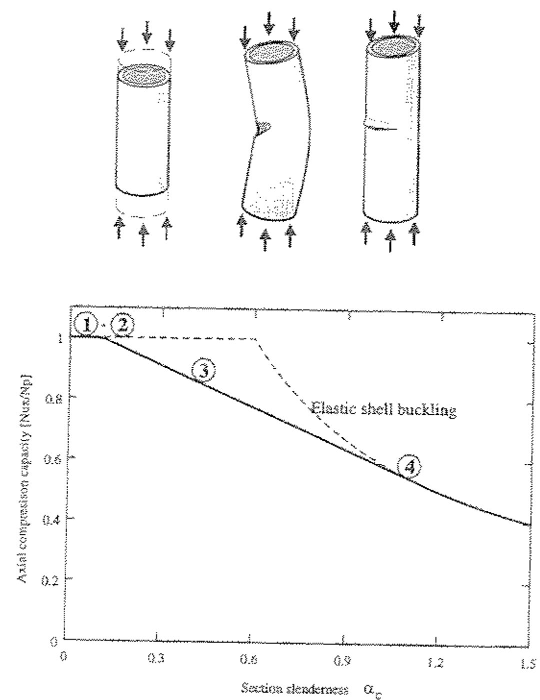


Figure 4-7 Section capacity - axial force

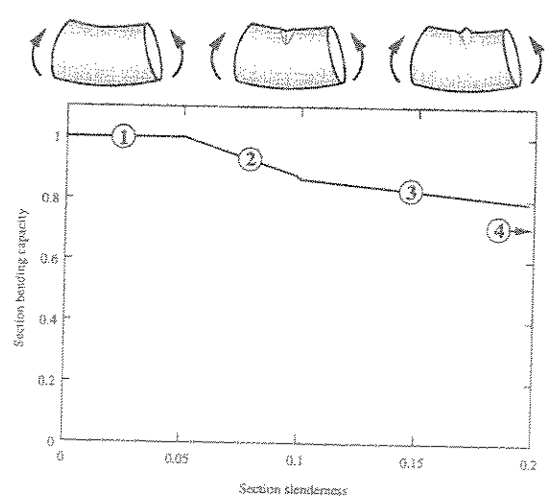


Figure 4-8 Section capacity – bending

4.2.5 Dented members

The cross-sectional capacity of a dented tubular may be expressed by e.g. /4/.

$$\frac{N_U}{N_P} = \exp(-0.08\,\frac{dd}{t})$$

and

$$\frac{M_U}{M_P} = \exp(-0.06\,\frac{dd}{t}) \qquad (4.6)$$

where dd = dent depth
t = wall thickness

Formulae for stability check are then expressed on a format similar to the conventional member stability check (equation (4.2)). /4/.

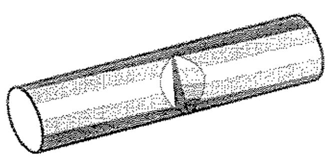


Figure 4-9 Idealised model of dented member

4.2.6 Effect of hydrostatic pressure

The major effects of external hydrostatic pressure are

- Hoop buckling, i.e. elastic or inelastic buckling of the shell wall between restraints
- Capped-end axial compression forces
- Reduction of axial capacity due to the presence of hoop stresses
- Reduction of bending capacity due to hoop stresses
- Accelerated load-shedding / more brittle post-ultimate behaviour.

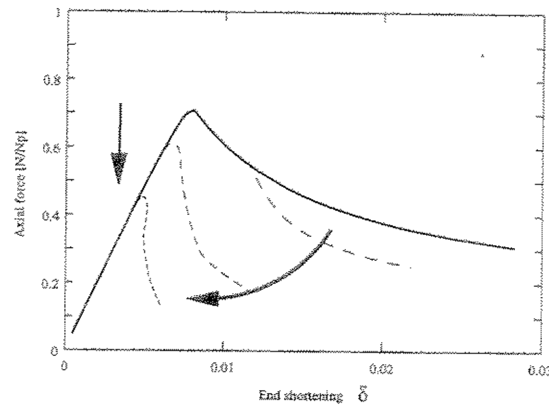


Figure 4-10 Effects of hydrostatic pressure

Hoop buckling

The hoop stress in a tubular member is given by

$$f_h = \frac{p\,D}{2\,t} \qquad (4.7)$$

where p is the external hydrostatic pressure.

The capacity with respect to hoop buckling is expressed by e.g. /4/.

$$f_{hU} = \begin{cases} f_y & 2.44 f_y < f_{he} \\ 0.7\, f_y \left(\dfrac{f_{he}}{f_y}\right)^{0.4} & 0.55\, f_y < f_{he} \le 2.44\, f_y \\ f_{he} & f_{he} \le 0.55\, f_y \end{cases} \qquad (4.8)$$

where
f_{hU} = ultimate hoop stress
f_{he} = elastic hoop buckling stress
$= 2\,C_h \dfrac{E\,t}{D}$

$$C_h = \begin{cases} 0.44\,\dfrac{t}{D} & 1.6\,\dfrac{D}{t} \le \mu \\ 0.44\,\dfrac{t}{D} + \dfrac{0.21\left(\dfrac{D}{t}\right)^3}{\mu^4} & 0.825\dfrac{D}{t} < \mu \le 1.6\dfrac{D}{t} \\ \dfrac{0.737}{\mu - 0.579} & 1.5 \le \mu \le 0.825\dfrac{D}{t} \\ 0.80 & \mu < 1.5 \end{cases}$$

$\mu = \dfrac{L}{D}\sqrt{\dfrac{2\,D}{t}}$

L = length of tubular between stiffening rings, diaphragms or end connections

Axial and bending capacity

The reduction in cross-sectional axial capacity and bending capacity may be expressed by e.g. /4/

$$\frac{N_{hU}}{N_P} = \sqrt{1+0.09\left(\frac{f_h}{f_{hU}}\right)^2 - \left(\frac{f_h}{f_{hU}}\right)^{2\eta}} \text{ , for } \frac{f_h}{f_{hU}} \le 1$$

$$\frac{M_{hU}}{M_P} = \sqrt{1+0.09\left(\frac{f_h}{f_{hU}}\right)^2 - \left(\frac{f_h}{f_{hU}}\right)^{2\eta}} \text{ , for } \frac{f_h}{f_{hU}} \le 1 \qquad (4.9)$$

where $\eta = 5 - 4\frac{f_y}{f_{hU}}$

Column buckling

The capped-end axial forces do not cause column buckling, but contributes to earlier yielding and in that way reduces the buckling strength of the member.

The member's compression capacity in presence of hydrostatic pressure may then be expressed on a format similar to the conventional member stability check, but with correction factors accounting for the presence of hydrostatic pressure /4/:

$$\frac{P_{Cr}}{N_P} = \begin{cases} \frac{1}{2}\left(1-0.28\bar{\lambda}_k^2 - 2\frac{N_q}{N_P} + \sqrt{\left(1-0.28\bar{\lambda}_k^2\right)^2 + 1.12\bar{\lambda}_k^2\frac{N_q}{N_P}}\right) & \text{for } \bar{\lambda}_k\left\{1-2\frac{N_q}{N_P}\right\}^{-1/2} < 1.34 \\ \frac{0.9}{\bar{\lambda}_k^2} & \text{for } \bar{\lambda}_k\left\{1-2\frac{N_q}{N_P}\right\}^{-1/2} \ge 1.34 \end{cases} \qquad (4.10)$$

where

N_q = capped-end axial compression force

$= \pi D t \left| 0.5 f_h \right| = \pi\frac{D^2}{4}p$

Post-ultimate behaviour

The presence of hydrostatic pressure leads to more 'brittle' post-ultimate behaviour of the member. Local buckling will occur for lower *D/t*-ratios than members in air (Section 4.2.4) and the local buckle will develop more rapidly once it is formed. This is illustrated in Figure 4-10.

4.2.7 Ductility limits

Tensile fracture is defined as de-connection of the member due to excessive tensile straining. This will be influenced by cracks, welds, residual stresses, stress or strain concentrations etc.

Most normalised steels used in offshore platforms show rather good ductility in tensile coupon tests with ultimate strain values larger than 15-20%. Geometric notches are introduced in the members when they are welded together, but overmatch weld material generally shield the defect regions. As long as there are no cracks present in the structure it should be assumed that the detail can achieve the strain level required to develop a plastic hinge.

Recent investigations /6/ suggest a limit of 5% nominal tension strain averaged over a length of maximum 20 times the thickness. This applies to welded connections without cracks, provided that fabrication is performed such that overmatch welds are achieved. Nominal strain is defined as the strain derived from a beam element model without including any stress concentration factor.

For structures with cracks the strains in the cracked region should not exceed the critical strain for the crack as defined by a fracture mechanics assessment. Alternatively, the critical strain for the crack could be imposed as a limiting strain (fracture criterion) in the non-linear analysis.

4.2.8 Cyclic degradation

The behaviour of tubular members tested under cyclic loading is shown in Figure 4-11. After global buckling the member is subjected to two cycles with large mean displacement before being straightened out and cycled at $\pm 2 \cdot \delta_{yield}$. Local buckling occurs during the first half-cycle. Note that the member attains its full tensile strength even after two severe compression cycles. However, the compression capacity is reduced after the first cycle. After three more cycles a through-thickness crack develops in the local buckle, and the capacity both in tension and compression drops rapidly. Failure is caused by through-thickness cracking in the most highly stressed area. This illustrates that tubular members may exhibit significant resilience even if they are loaded significantly in excess of the conventional (buckling) failure criterion.

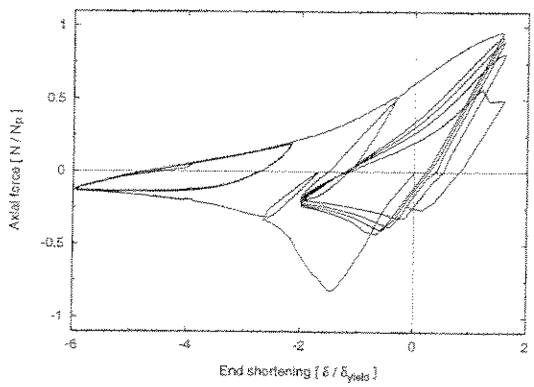


Figure 4-11 Cyclic load-displacement relationship for a tubular beam-column

Recent investigations /13/ indicate that a conservative low cycle fatigue criterion is given by the AWS curve (1983), converted to strains and extrapolated into the low-cycle regime.

$$\Delta\varepsilon = 0.055 \cdot N^{-0.4} \tag{4.11}$$

In addition, an upper bound on the strains is defined by the monotonic local buckling criterion. Ref. Figure 4-12.

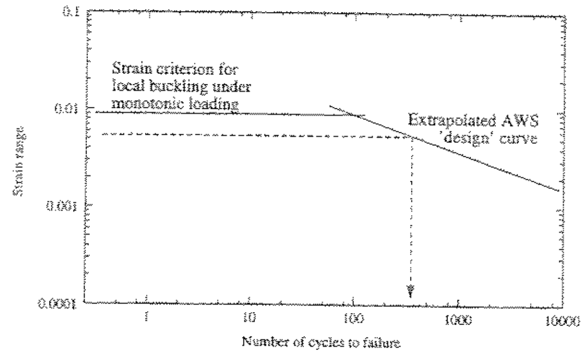


Figure 4-12 Low cycle fatigue criterion for cyclic loading of tubular beam-columns

Figure 4-13 Fracture of tubular members

4.3 Tubular joints

4.3.1 General

In elastic analysis of tubular framed structures, it is conventional practice to assume the joints form rigid connections. Compliance with code checks on capacity ensures that this assumption is reasonable.

Under extreme loads, the non-linear deformations of the joint and failure characteristics, can influence the disposition of forces and the overall structural response.

Failure of tubular joints generally involves some combination of the following local and global modes as illustrated in Figure 4-14:

1. Local plastic deformation (yield) of the chord around the brace intersection
2. Cracking in the chord at the weld toe (and propagation to severance)

3. Local buckling in compression areas of the chord
4. Ovalisation of the chord cross-section
5. Beam shear failure across a gap K joint chord
6. Beam bending of the chord under T/Y action.

The specific response depends on:

- the type of joint (T/Y, X, K; simple, stiffened, grouted; etc)
- the loading (Axial - tension/compression; bending - in-plane bending/out-of-plane bending etc)
- the joint geometry parameters (β, γ etc)

Figure 4-15 and Figure 4-16 compare typical load-deformation responses for axially loaded joints as seen in isolated component tests performed with idealised boundary conditions.

Examples of appropriate response characteristics are given in the Commentary. Figure 4-17 defines the non-dimensional expressions for geometry used to describe joint response characteristics and modelling in Chapters 4 and Chapter 5.

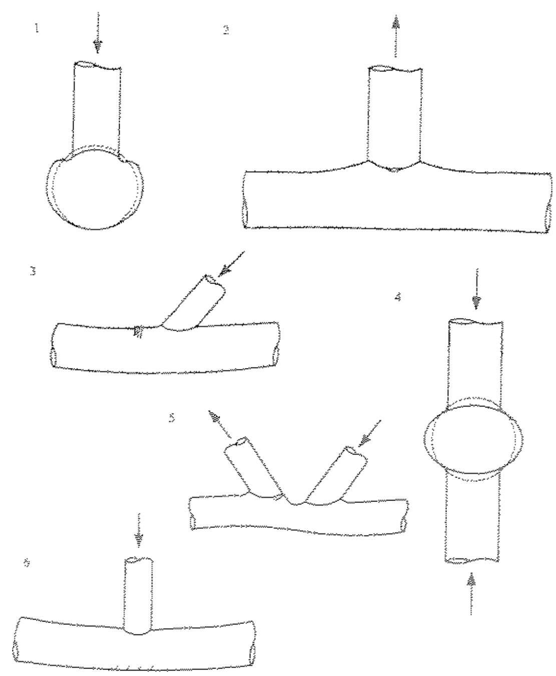


Figure 4-14 Tubular joint failure modes

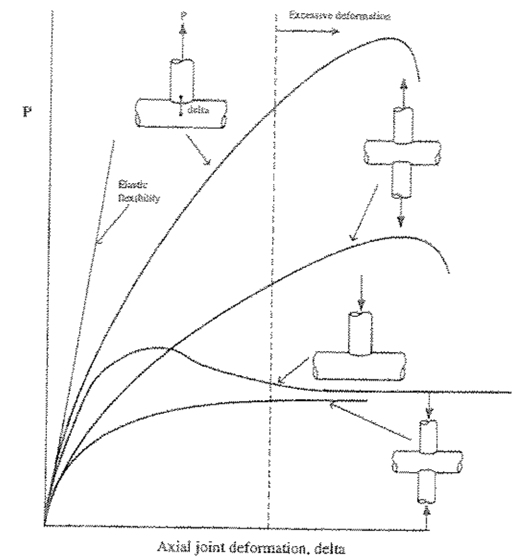


Figure 4-15 Load-deformation responses for simple T and X tubular joints

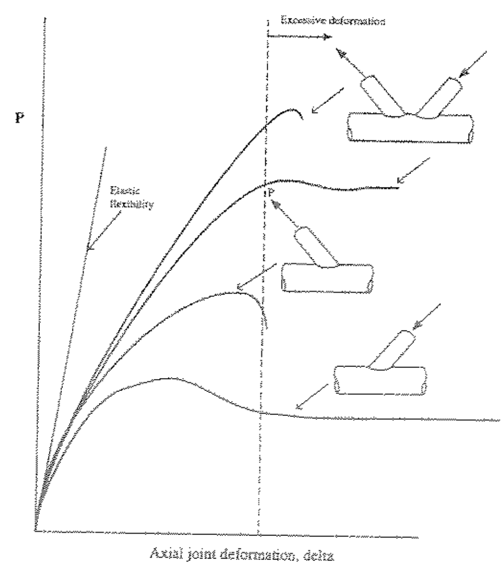
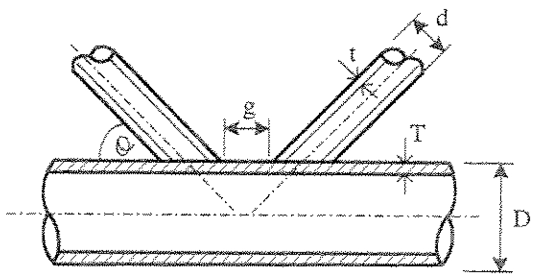


Figure 4-16 Load deformation responses for Y and K simple tubular joints



$\beta=d/D$; $\gamma=D/2T$; $\tau=t/T$; $\zeta=g/D$

Figure 4-17 Tubular joint non-dimensional geometry definitions

The behaviour of tubular joints depends on the complex interaction of loading and geometric parameters. However general observations can be made:

4.3.2 Compression joints

Compression joints generally exhibit a softening response characteristic with buckling and/or plastic deformation of the chord wall. Local buckling is associated particularly with high γ and large β.

4.3.3 Tension joints

Tension joints generally suffer large plastic deformation around the intersection, precipitating chord wall cracks in the hot spot region. The joint can sustain increasing monotonic load as the crack propagates, until gross separation of the brace and chord occurs. At this points load is shed rapidly giving an overall brittle response characteristic and a low residual capacity.

4.3.4 K joints - shear

Balanced axial loading of eccentric K joints (tension-compression) generates shear across the gap region. For high β joints, failure of the chord wall section can occur in shear, or may involve initial compression failure at one intersection followed by tension cracking at the crown of the other. The response mode has a brittle characteristic with rapid load shedding. At low β plastic deformation of the chord around the compression intersection may dominate and exhibit a ductile response.

For overlap K joints, shear failure may occur along the common brace weld intersection in combination with limited plastic deformation of the chord. There is also potential for local buckling failure of the brace in the intersection region.

4.3.5 In-plane moment

Under in-plane bending, cracking generally initiates in the chord wall at the tension crown of the intersection with buckling of the chord wall on the compression side.

4.3.6 Out-of-plane moment

Under out-of-plane bending local buckling of the chord wall occurs at the compression saddle, reducing stiffness with crack initiation and fracture at the tension saddle eventually limiting the capacity.

4.3.7 Ring-stiffened joints

Ring stiffeners limit the degree of chord wall ovalisation thereby increasing joint capacity and stiffness. It would be rare for a ring stiffened joint to warrant explicit modelling in ultimate strength analysis. Response characteristics should be based on a case-by-case assessment, with due regard to local hot spots under tension in the vicinity of the brace-ring intersections.

4.3.8 Grouted joints

A grouted core stiffens and strengthens a tubular joint in compression and explicit modelling in ultimate strength analysis is not likely to be required as the capacity of incoming members will govern. Under tension, particularly for high β, the joint is less compliant than its ungrouted counterpart and cracking leading to severance may occur. In the absence of data, ungrouted tension joint characteristics may be adopted. Alternatively the capacity in tension may be checked by determination of the shear capacity of the chord wall along the brace to chord intersection line.

4.3.9 Joints with cracks

Analytical and experimental results indicate that the collapse load of cracked tubular joints can be predicted by multiplying the capacity of the uncracked joint by an area reduction factor, F_{AR}. The reduction factor is given in /15/ as:

$$F_{AR} = (1 - \frac{A_C}{A}) \cdot \left(\frac{1}{Q_\beta} \right)^{m_q} \tag{4.12}$$

Q_β is the tubular joint geometry modifier, and m_q is the power allocated to Q_β. For a further description reference is made to the Commentary and to /15/.

4.3.10 Cyclic loading

Joint failure is generally preceded by significant non-linear deformation and it is therefore to be expected that the response would be modified by cyclic loading at load levels approaching the capacity limit, e.g.:

- high stress low cycle fatigue cracking (ductility exhaustion)
- increasing deformations and reducing capacity (incremental collapse)

Only limited tests are available for individual cases, however, indicative guidance on ductility limits is given in the Commentary. Appropriate consideration should be given to cyclic loading effects in the application of results from ultimate strength analysis.

4.3.11 Frame effects

Response characteristics are derived from isolated tests of simple joints under an applied loading mode. In a real structure, a joints is generally constrained in a plane, brace and chord loads coexist, deformations are limited within a panel and the force/moment demand and interaction is influenced by the resistance.

Only limited tests have demonstrated joint failure in a frame and data are insufficient for definitive guidelines. However, calculated responses should be reviewed to ensure they are realistic in the confines of a framed structure.

4.4 Frame behaviour

4.4.1 General

A frame structure reaches its ultimate capacity when a sufficient number of plastic hinges has formed, or a sufficient number of components have passed their capacity limits such that the structure turns into a kinematic mechanism. This is often denoted as systems collapse.

For most jacket structures, failure of the diagonal bracing (either compression buckling or tension yielding) within a single bay will cause the major load carrying mechanism to change from a dominating truss-work action to a dominating portal frame action. The post-collapse region is characterised by more or less rapid load shedding, with the displaced topside weight as additional driving force. From a numerical point of view, the structure can only maintain equilibrium at reduced load levels.

In non-redundant structures, systems collapse coincides with first member failure. The failure mode tends to be 'brittle', i.e. first member failure is followed by immediate loss of capacity. A ductile 'frame' mechanism forms at a reduced load level.

Most redundant framing systems show collapse strength higher than, or equal to, first member failure. Subsequent behaviour is ductile with significant energy dissipation before major loss of capacity occurs. This is illustrated in Figure 4-18. The behaviour above refers to structures with rigid (i.e. strong) connections. Yielding at joints may 'shield' incoming members from further loading and thus alter the structural behaviour

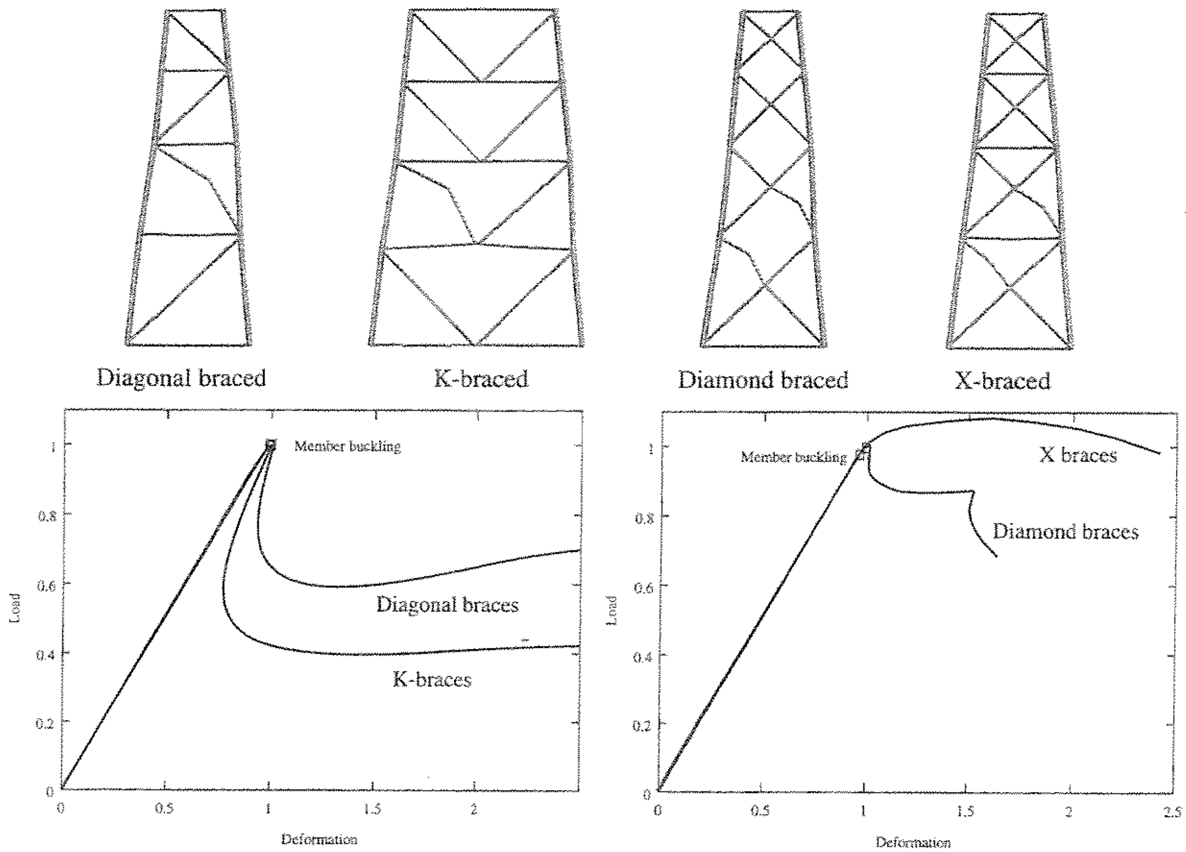


Figure 4-18 Load - deformation characteristics for different bracing systems

significantly, as illustrated in Figure 4-19 both for limited and unlimited joint ductility.

4.4.2 Cyclic behaviour

One measure of the system strength is that obtained from static 'pushover' analyses. This strength provides a measure of the capacity of the structure when subjected to the forces from one single large wave and the associated wind and current effects. In a severe storm, however, several large waves will be generated and the limit state may be reached after a number of cycles of e.g. crack growth or local buckling.

Structures exposed to cyclic loading should be checked against possible cyclic degradation due to repeated loading.

However, recent research projects /11/ /12/ indicate that cyclic degradation has little practical consequence for fixed offshore structures under wave loading.

Members and joints undergoing cyclic inelastic strains should be subjected to separate checks as indicated in Sections 4.2.8 and 4.3.10. Further discussion is included in the Commentary.

Tubular members loaded only in the pre-buckling range will not be susceptible to cyclic degradation during extreme storm loading. Thus, if the structure is not utilised beyond first member buckling, no further assessment of cyclic member behaviour should be required.

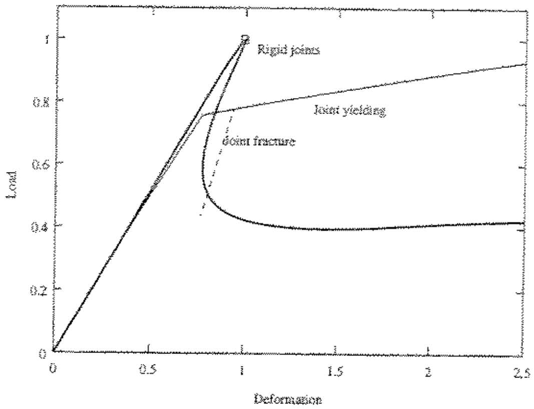


Figure 4-19 Structural behaviour with joint yielding and limited joint ductility

4.5 Piles

4.5.1 Lateral soil failure

Lateral soil failure occurs when the applied lateral environmental loading exceeds the soil and pile's ultimate lateral capacities, provided this occurs before the space frame reaches its ultimate strength. The ultimate collapse strength is achieved immediately after soil failure, when a portal frame mechanism develops in the piles below mud-line; plastic hinges occur just below the jacket at mud-line and at the point of maximum bending moment some distance below mud-line.

The lateral soil capacity may be given by e.g.

$$p_u = \begin{cases} 3 \cdot c & , \text{at } z = 0 \\ 9 \cdot c & , \text{for } z \geq z_R \end{cases} \tag{4.13}$$

for cohesive soils. Intermediate values are determined by linear interpolation. c is the undrained shear strength of the soil and z_R is the depth of the reduced resistance zone, given by:

$$z_R = \frac{6D}{\frac{\gamma D}{c} + J} \tag{4.14}$$

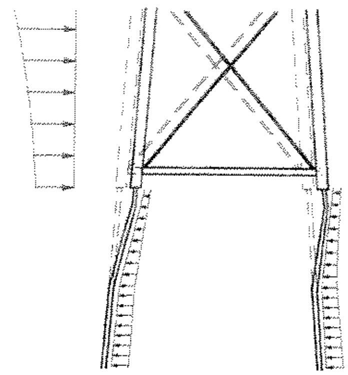


Figure 4-20 Soil lateral failure

Here D is the pile diameter, γ is the effective unit weight of soil and J is an empirical constant $\frac{1}{4} \leq J \leq \frac{1}{2}$.

The soil behaviour is then given by non-linear P-δ curves describing the soil behaviour up to and past the ultimate strength.

4.5.2 Punch through failure or pull out failure

Punch through or pull out failure occurs when the applied lateral environmental loading results in pile axial loads exceeding their ultimate capacities in compression and/or

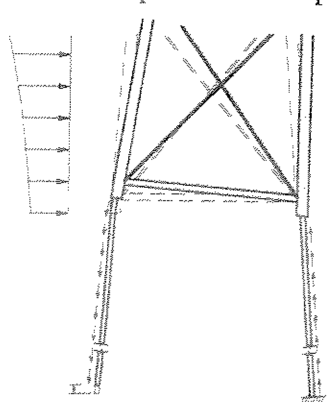


Figure 4-21 Soil axial failure

tension. In the case that the steel framework is sufficiently strong, or foundation is relatively weak, the ultimate capacity is achieved when piles punch through and pull out from the soil. A clear failure mechanism is formed with the steel frame rotating as a 'rigid body' about a neutral 'rotation axis' passing through the heads of those piles which have not exceeded their axial capacities.

The axial skin friction is generally given by e.g.

$$f = \alpha \cdot c$$

where

$$\alpha = \begin{cases} \frac{1}{2}\left(\frac{c}{p_0}\right)^{-1/2} & \text{, for } \frac{c}{p_0} \le 1 \\ \frac{1}{2}\left(\frac{c}{p_0}\right)^{-1/4} & \text{, for } \frac{c}{p_0} > 1 \end{cases} \quad , \alpha \le 1 \tag{4.15}$$

for cohesive soils. Here c is the undrained shear strength of the soil and p_0 is the effective overburden pressure. The soil behaviour is then given by non-linear T-z curves describing the soil behaviour up to and past the ultimate strength.

4.5.3 Combined soil-structure failure

In the case that the steel framework and the foundation have similar strengths a combined framework-foundation failure mechanism may occur, where the failure mechanism of the frame may be influenced by redistribution of pile loads.

4.5.4 Cyclic degradation

Cyclic loads cause deterioration of the lateral bearing capacity as indicated in Figure 4-22. The soil capacity and the non-linear P-δ characteristics given in most codes represent the fully degraded properties of the soil. The capacity under monotonic, static loading can be significantly higher, as shown in Figure 4-23.

4.5.5 Loading rate effects

High loading rates (compared to laboratory test loading) can also contribute to increase the capacity /8/. In static loading tests the loading rate is usually less than 10% of the ultimate capacity per hour. At wave loading

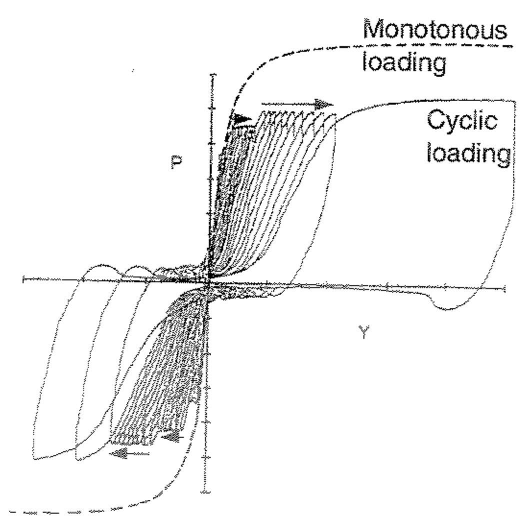


Figure 4-22 Cyclic soil degradation

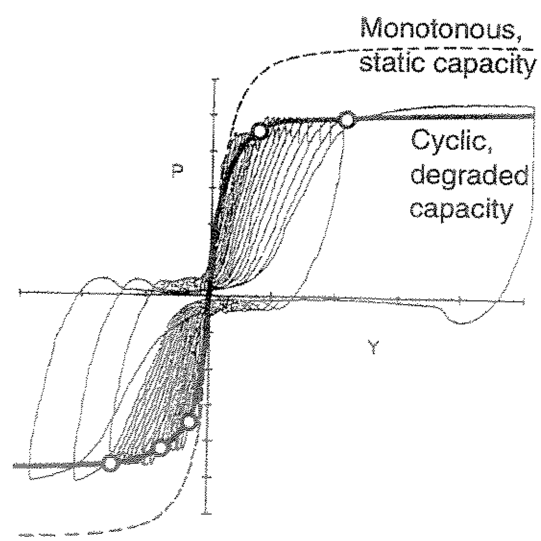


Figure 4-23 Cyclic degraded soil properties

the loading rate will be about 10 000 times faster.

An increase in capacity of some 40% for lateral loading and 50% for axial loading is suggested for wave loading.

However, it should be noted that any increase in strength due to rate effects only apply to the dynamic (variable) part of the soil reaction. If 60% of the soil reaction results from dead loads and live loads, rate effects can only be ascribed to the remaining 40% that are due to environmental loading. This is illustrated in Figure 4-24.

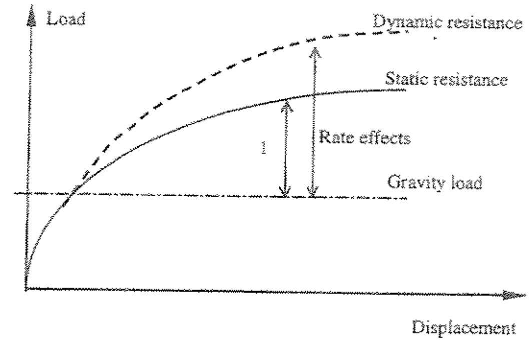


Figure 4-24 Loading rate effects

5 Structural Modelling

5.1 General

The structures must be modelled in sufficient detail to ensure that the non-linear analysis program adequately captures the relevant global and local failure modes and load redistribution.

The models for component strength, such as member compressive strength and joint strength are semi-empirical. They have a theoretical basis, but are formulated to conform to experimental data. In general, all theoretical formulations need some calibration in order to represent the behaviour of 'real' structural components with sufficient accuracy.

Moreover, it should be possible for an engineer to select specific failure criteria (e.g. a specific code of practice) and have the analysis tools calculate the structure's strength based on those criteria. In such a case, the requirement for the analysis tool is not to present theoretically correct solutions for the structure, but rather to present a consistent strength estimate based on the engineer's specifications.

This implies that the analysis tools should be able to represent different failure criteria, from the theoretical, 'ideal', solution to characteristic lower bound solutions as specified by different codes of practice. Some non-linear analysis programs have built-in features to calibrate component failure modes to specific criteria. For other programs, the engineer must give special consideration during the modelling of the structure in order to make the program represent the required failure modes or limiting criteria. Which considerations to take depends on the component (member / joint / foundation) and on the mathematical formulation.

If there is any doubt about the FE formulation, a simple model should be subjected to a well-defined load and deformation path. This will allow the results to be judged and calibrated against engineering practice.

Instead of modelling the structure out of purely geometric considerations, the modelling must consider the analysis tool that will be used for the non-linear analysis, and the mathematical formulation that is embedded within the program.

This section gives a set of modelling recommendations to help make the non-linear analysis tool produce reliable results that conform to recognised failure criteria and design formulations.

Load modelling is treated in Chapter 6, while Chapter 7 discusses the actual analysis execution.

5.2 Frame modelling

The space frame model should describe the three dimensional geometry of the platform.

The model for ultimate strength assessment can usually be significantly simpler than models required for design and fatigue analysis. The primary framework, essential in maintaining overall integrity of the structure for the in-place condition must be included in the structure model. Secondary structures and members generating dead loads and / or environmental loading need only be represented in sufficient detail to introduce the relevant loads on the primary structure.

The analytical models should consist primarily of beam elements. The structural members of the framework may be modelled using one or more beam elements for each span between the nodes of the model of the framework.

5.2.1 Primary framework

The primary framework of the structure comprises those members that provide the stiffness and strength of the structures. These are usually the legs, the piles, the vertical diagonal members and the main plan frame bracing members.

5.2.2 Secondary framework

The secondary framework consists of members that do not contribute to global stiffness and strength of the framework. In general, their structural contribution may be neglected and they need not be included in the model as structural members. Boat-landing / fenders, spider-deck, walk-ways etc. are examples of secondary members.

Secondary framework should be modelled in sufficient detail to transfer the required loads into the primary framework.

Some local over-stresses in secondary framework should be accepted if they occur in areas where the model has been purposely simplified and where the adjacent, primary,

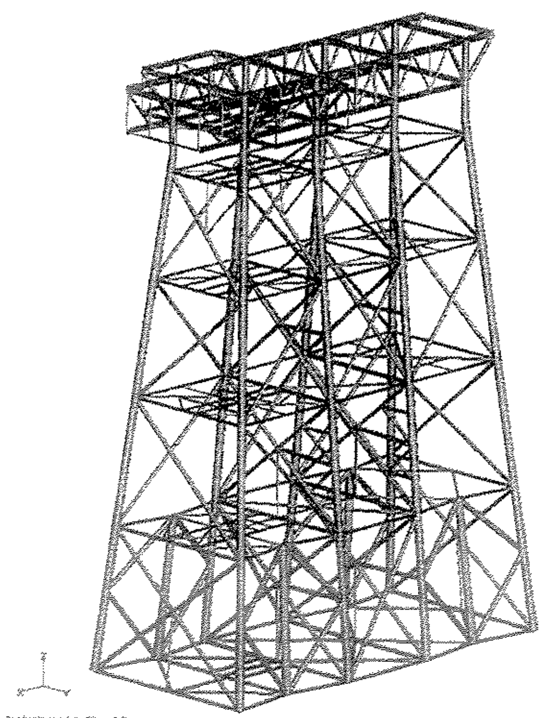


Figure 5-1 Primary framework.

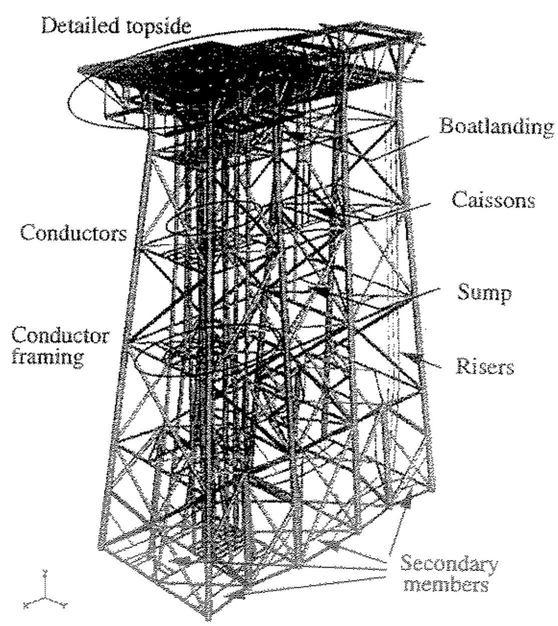


Figure 5-2 Secondary framework

members show sufficient capacity. This should however be subject to separate justification in each case.

The following secondary framework should be included in the model:

i) Members / joints which are essential for transfer of reaction loads from conductors and appurtenances etc. to the main structural elements.

ii) Members / joints which are highly loaded by local wave action

Alternatively, a separate assessment may be done on the local behaviour. The global assessment may be performed with a simplified model if it is demonstrated that the load can be carried and transferred by the secondary framework.

Secondary members associated with launch framing, mud-mats, conductors support during transportation etc., should be included in the model if they provide significant support to primary members and thus contribute to the system's capacity.

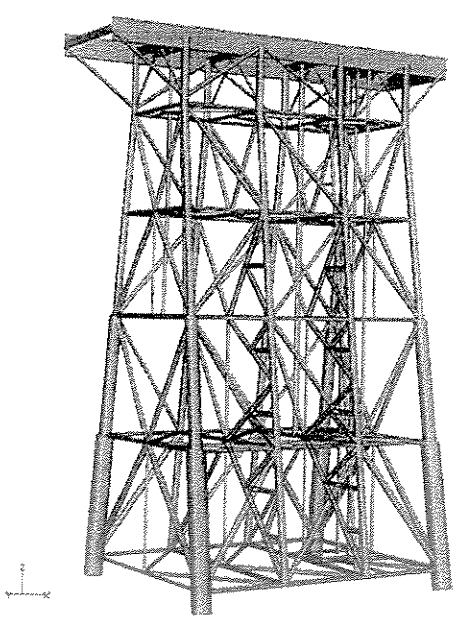


Figure 5-3 Launch frames contributing to the structural strength

When neglecting the structural contribution of secondary members, their load attracting properties, that is loading due to self-weight and or hydrodynamic loading, should still be accounted for and included in the appropriate loading condition.

Conductors and other appurtenances, such as launch cradles, mud-mats, J-tubes, risers, skirt pile guides etc. should be included in the model if they contribute significantly to the overall strength of the structure or foundation.

Otherwise, they may be disregarded as structural elements.

5.2.3 Deck structure

The stiffness of the deck structure should be modelled in sufficient detail to adequately represent the deck jacket interface, such that the applied topside loading and structural self-weight is appropriately distributed to the sub-structure framework. An example is shown in Figure 5-6. This modelling introduces artificial split forces in the MSF, and should only be used in assessment of the sub-structure.

In situations where the deck is giving significant contribution to the strength of the jacket, the deck members should be included as structural members, or subjected to a separate yield check.

5.2.4 Foundation modelling

The foundation should be modelled and analysed as a fully integrated part of the structure using non-linear soil P-y and T-z curves representing the soil stiffness and capacity. Particular care should be taken when modelling relatively thin layers near the mud-line, where P-y curves change rapidly,

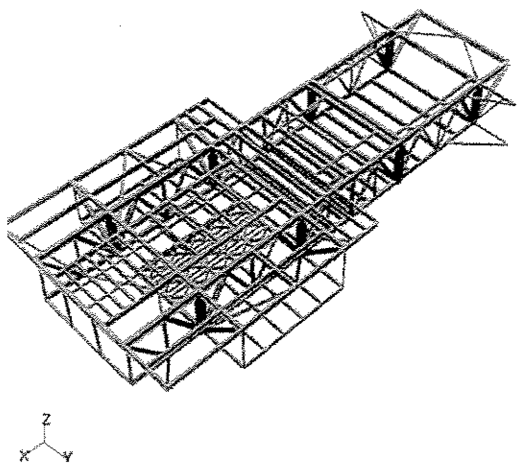


Figure 5-4 Detailed modelling of deck structure

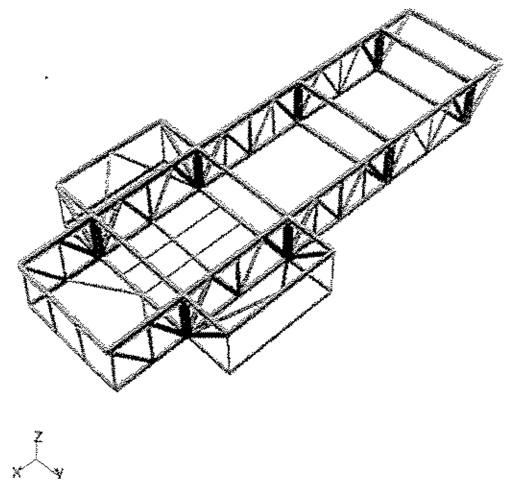


Figure 5-5 More suitable modelling of deck structure

and to accurately model the soil layers near the bottom end of the piles.

The effects of global seabed scour and local scour in granular soils, and the partial loss of soil-pile contact in cohesive soils should be accounted for.

When modelling the individual piles in a pile group, non-linear soil P-y and T-z curves have to be adjusted to account for pile group effects. The influence of a pile group on global structural behaviour may be modelled by simpler means, such as the use of an equivalent single member with the equivalent structural and foundation properties, taking due account of pile group effects.

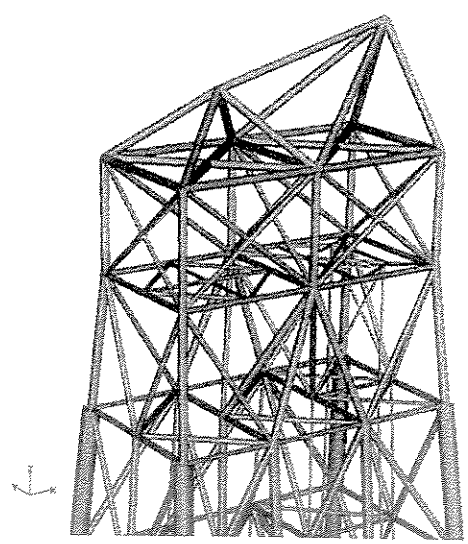


Figure 5-6 Deck modelling retaining stiffness and mass properties of MSF and topside

5.2.5 Pile connectivity

The sliding action of piles within legs should be modelled with the approximate constraint conditions, which allow unrestrained differential axial displacement and rotations, but couple the lateral displacements of piles and legs.

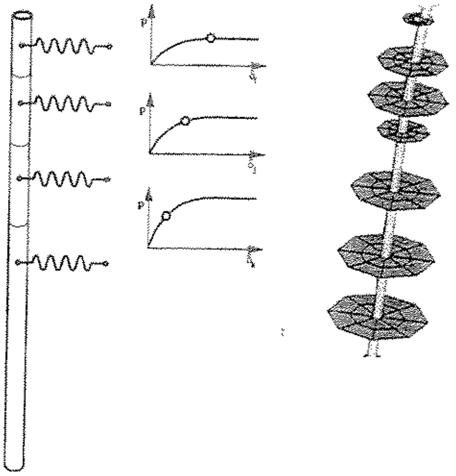


Figure 5-7 Pile-soil modelling

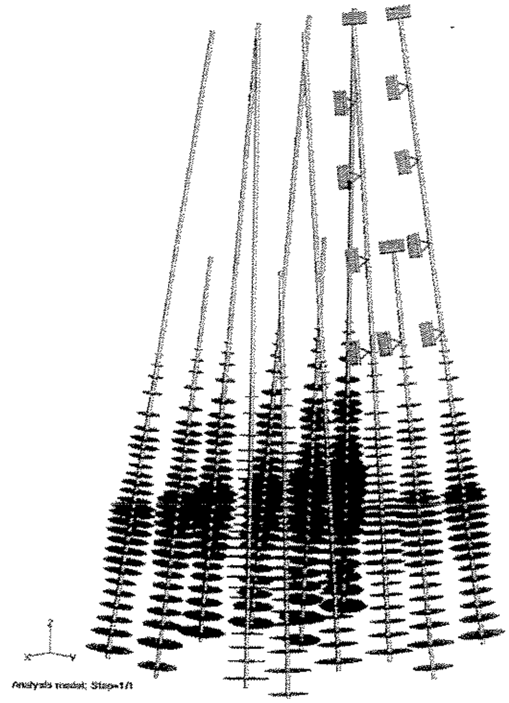


Figure 5-8 Foundation modelled by non-linear soil springs

5.2.6 Grouted piles

Grouted piles can be modelled as composite leg-pile members. Alternatively, leg and pile members can be modelled as separate elements with full coupling between end degrees of freedom.

For un-grouted piles, the lateral displacements for pile and leg are usually coupled at each horizontal elevation, but some differential movement can occur between elevations.

5.2.7 Conductors

Conductors provide limited contribution to the strength of the steel frame system. In most cases, they can be modelled as pure load-attracting members; as long as their load contributions are correctly captured, the conductors can be omitted from the strength calculations.

However, for structures with limited foundation resistance, conductors can contribute significantly to the foundation stiffness and collapse strength of the structure. In that case, the conductor should also be modelled and analysed as a structural element and included in the integrated structure soil model. The conductor guide framing at mud-line may then be highly loaded and may need a more detailed inclusion in the structural model as primary framework.

5.2.8 Conductor connectivity

Conductor guides should be modelled in sufficient detail to transfer the required loads into the primary framework. Local over-stresses in conductor guides should not cause concern if they occur in areas where the model has been purposely simplified, and the surrounding primary members have sufficient capacity.

If conductors are included as structural members the sliding action of conductors within the guide frames should be modelled. The constraint conditions should allow for unrestrained differential vertical displacement but couple the lateral displacements of conductors and guide frames. Differential rotations should also be unrestrained.

The contact action between curved conductors and their guide frames due to the imposed conductor curvature and friction may require specific consideration.

5.2.9 Joint eccentricities

Joint eccentricities which are less than D/4 need not be included in the structural model (D = the can diameter).

Elastic joint flexibility reduces the rotational restraint on members in frames, and thereby reduces the buckling strength of the members. However, recent studies /10/ indicate that elastic joint flexibility has minor influence on the collapse capacity of tubular frames, and that conventional (centre-centre / rigid joint) modelling gives conservative estimates of first member failure and collapse capacity for framed offshore structures. Face-to-face modelling with flexible joints give some benefit (with respect to increased capacity estimates), but the benefit must be weighted against the increased complexity in modelling.

For large diameter chords or short braces, however, local joint stiffness should be considered.

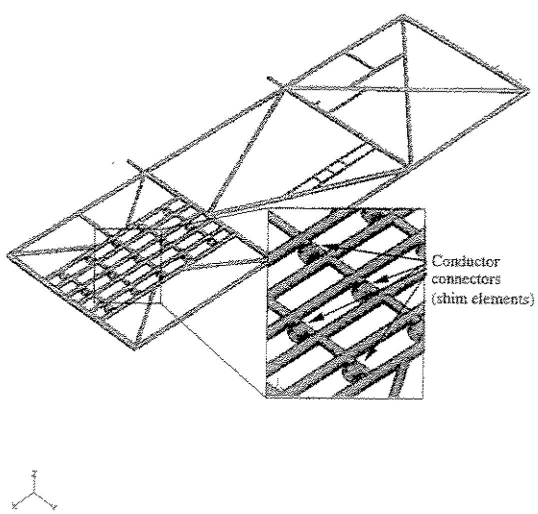


Figure 5-9 Detailed modelling of conductor framing

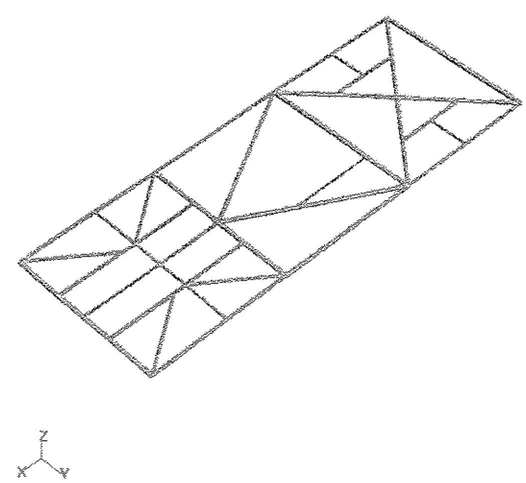


Figure 5-10 Conductor framing modelled in sufficient detail for analysis of primary framework

5.2.10 Grouted joints

The strength of a grouted joint is likely to exceed that of the connected members. The grouting will also significantly increase the rotational restraint imposed by the joint, and thereby increase the buckling capacity of the connected member(s).

One modelling technique to represent the increased joint strength and stiffness is by rigid links from chord centre to the face of the chord.

5.2.11 Cracked joints

Damaged components should be modelled in sufficient detail to assess the impact of the damage on the global behaviour of the structure.

The area of the crack is usually quite small compared to the total brace / chord footprint area. Also, a joint is in itself a redundant system that allows for significant redistribution of stresses from the most highly stressed area.

A lower bound on the remaining structure's strength can usually be obtained by removing the affected joint(s)/member(s) from the model. A less conservative estimate is obtained by reducing the strength of the affected joint by some fraction, as indicated in 4.3.9

The presence of a crack will also limit the ductility and the cyclic capacity of a joint. These properties should be evaluated by refined analysis of the joint in question if the analysis indicates that a cracked joint is loaded up to its static capacity.

5.2.12 Ground joints

Grinding is a common procedure for repair of fabrication and / or fatigue cracks found during inspection of the structure. If the ground area is small compared to the total brace / chord footprint area it can reasonably be assumed that the grinding of one or more joints will have limited effect on the overall systems behaviour, and can be neglected in the global frame model.

If a significant fraction of the wall thickness has been removed, the flexibility and capacity of the joint may have to be assessed in more detail. A lower bound on the remaining structure's strength can usually be obtained by removing the affected joint(s)/member(s) from the model. A less conservative estimate is obtained by reducing the strength of the affected joint by some fraction, preferably established through refined analysis of the joint in question.

5.2.13 Dented members

Damaged components should be modelled in sufficient detail to assess the impact of the damage on the global behaviour of the structure.

A lower bound on the remaining structure's strength can usually be obtained by removing the affected member(s) from the model. A less conservative strength estimate is obtained by modelling the damage in the non-linear analysis.

Some non-linear analysis tools include special formulations to model dented / distorted members. Alternatively, the damage can be modelled explicitly by shell elements, or a reduced cross-sectional area can be specified in the damaged zone.

5.2.14 Grouted members

Grouting is a simple repair method to bring the strength of damaged members back to, or in excess of, the initial member strength. The strength of grouted members can conservatively be set to the initial member strength. However, since additional stiffness attracts additional forces, any additional stiffness caused by the grouting should be included in the model. Gross distortions (out-of-straightness) of the grouted member needs to be taken into account.

The grouted member can be included in the FE model, either as a composite member, or as a steel member with thickness adjusted to give equivalent member properties.

5.2.15 Yield strength

For ultimate limit state assessment of new designs, nominal (or specified minimum) yield strengths should be applied. For reassessment of existing structures some codes accepts the use of actual (coupon test) or expected mean yield stress.

5.2.16 Strain hardening

Increased strength due to strain hardening should not be acknowledged. For compression members, strain hardening will only occur in the post-collapse range and reduce the load shedding. For tensile members, ductility criteria will limit strains before strain hardening becomes significant.

For joints the strain hardening will to some extent be included in empirically based capacity criteria.

5.2.17 Strain rate effects

Strain rate effects should normally not be acknowledged.

5.2.18 Locked-in-forces

In most cases with ductile collapse modes, locked-in forces will not reduce the load-carrying capacity. In the event that locked-in-forces contribute to reduce the load-carrying capacity, they should be modelled.

5.2.19 Corrosion allowance

The modelling of corrosion allowance depends on available inspection data. If reliable inspection data indicate that the amount of corrosion is limited the full wall thickness of the member can be used. If not, the wall thickness should be reduced by the corrosion allowance.

5.3 Member modelling

5.3.1 General

Primary members should be modelled in such a way that they adequately reproduce the relevant beam-column failure modes described in Section 4. I.e.:

- Tension yielding
- Bending failure
  (single curvature and counter-curvature)
- Compression (crushing) failure
- Stability (buckling) failure
- Post-buckling load-shedding behaviour
- Interaction with local buckling

Members should normally be modelled with centre-to-centre length. Modelling with face-to-face length reduces the buckling length of the member and increases the apparent member strength. Elastic joint flexibility will on the other hand reduce the rotational restraint on the member. Recent investigations /10/ indicate that the effect of face-to-face modelling increases the capacity estimates by some 5%, whereas the elastic joint flexibility reduces the capacity estimates by some 2-4%. The overall effect of this modelling seems to be small; centre-centre modelling of members is generally recommended due to the simplicity and due to slightly conservative strength estimates.

5.3.2 FE selection

Phenomenological modelling

In some programs based on a phenomenological formulation, the type of failure behaviour of the element must be assessed prior to the analysis and the element type selected accordingly.

- Axially loaded members. These are members that are expected to undergo tensile yielding or buckling during ultimate strength analysis.
- Moment resisting members. These are members that are expected to yield during

the ultimate strength analysis, primarily due to high bending stresses.

- Frame members. These are members used to represent beam-column behaviour, i.e. a combination of non-linear bending behaviour and column buckling

The elements have been tailored to model column buckling or beam bending and therefore one FE is generally sufficient to model the desired member response.

Plastic hinge beam-column models

The elements embodied in this type of programs have specifically been derived to model beam column behaviour. This element formulation captures tension yielding, bending failure, compression failure and member stability (buckling) in the same finite element. No assessment of the element failure behaviour is necessary to select proper FE type.

These elements have been specifically derived to model beam-column behaviour and therefore each member span only requires one single element to model the buckling response.

General beam-column models

This element formulation captures tension yielding, bending failure and compression failure in the same finite element. However, these programs generally do not include beam elements that incorporate geometric non-linearity at the element level.

To model buckling response multiple beam elements are required along a member span. Four to six elements per member are recommended. One to two members should be sufficient for members failing in bending or tension yielding.

Shell modelling

Non-linear shell FE models of members capture all the above failure modes, provided the FE mesh accommodate for the relevant effects of the failure modes.

16 elements around the circumference of a tubular are in most cases found to be sufficient. Number of elements along the member should be determined by the length-to-width ratio of the elements, which should not exceed 4-5.

5.3.3 Material modelling

Material modelling in stress space (σ–ε) are of primary concern for general beam-column formulations or shell-type FE formulations.

Plastic hinge models use cross-sectional forces and moments (N, Q, M) to model the integrated behaviour of the cross-section. Thus, the material modelling is represented through an accurate representation of the cross-section behaviour.

Phenomenological element formulations use non-linear P-δ curves to represent the integrated behaviour of the entire member. Detailed material modelling is not applicable.

Recent investigations /6/ indicate that overall strains for buckling members are limited to 0.3-0.5%. This means that member strains generally are limited to the yield plateau of conventional structural steel. For compression members, strain hardening will only occur in the post-collapse range and reduce the load shedding. For tensile members, ductility criteria will limit strains before strain hardening becomes significant.

Therefore, increased strength due to strain hardening should normally not be acknowledged. Elasto-plastic material behaviour should generally be modelled by a bi-linear elastic-perfectly plastic relationship as indicated in Figure 5-11. In case of numerical problems a small strain hardening may be assigned as indicated in the Figure.

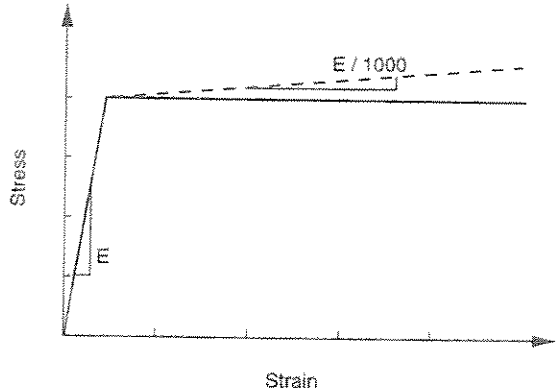


Figure 5-11 Elasto-plastic material modelling

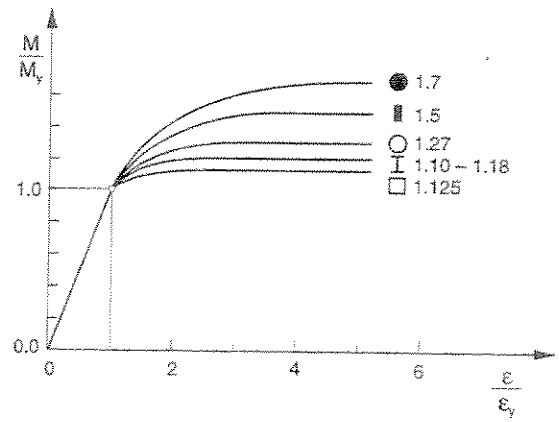


Figure 5-12 Section M-θ curves

5.3.4 Cross-sectional modelling

Special modelling of cross-section behaviour in force space (M, N, Q) is of primary concern for plastic hinge beam-column models and for some general beam-column formulations.

Phenomenological element formulations use non-linear P-δ curves to represent the integrated behaviour of the entire member. Specific modelling of the cross-section is not applicable.

General beam-column models describe the cross-section behaviour either by P-δ or M-θ curves for different cross-section types (Figure 5-12) or by integration of the material behaviour through integration points throughout the section (Figure 5-13).

In the former formulation, detailed cross-sectional modelling is possible through appropriate selection of the P-δ and M-θ curves.

In the latter formulation, the cross sectional behaviour is determined by the geometry and by the number of integration points throughout the section.

16 integration points are normally sufficient to model tubular sections.

Shell FE formulations the cross sectional behaviour is determined by the geometry and by the number of finite elements around the circumference of the section.

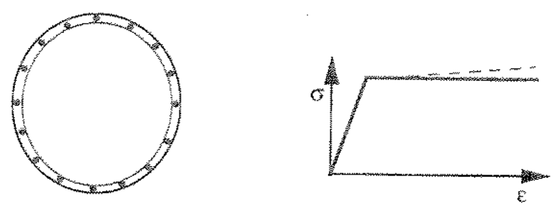


Figure 5-13 Integration of cross-section behaviour

Plastic hinge beam-column models

Plastic hinge models use cross-sectional forces and moments (N, Q, M) to model the behaviour of the cross-section. Interaction between axial force, bending moments and other force components are modelled by plastic capacity surfaces as illustrated in Figure 5-14.

One-surface plasticity models treat the cross-section as purely elastic until fully plastic. The plasticity model is only represented by the outer (bounding) capacity surface only.

Multi-surface formulations include a separate yield surface to represent first fibre yield and gradual plastification of the cross-section. The inner (yield-) surface denotes the demarcation of elastic behaviour. For force combinations in the intermediate region the cross-section is partially plastic. The bounding surface identifies the fully plastic capacity of the cross-section.

The conventional (linear) first yield criterion is indicated by the dotted line. Plastic flow theory requires the initial yield surface to have the same shape as the full plastic capacity surface. Thus, the first fibre yield condition may deviate from the conventional, linear yield condition.

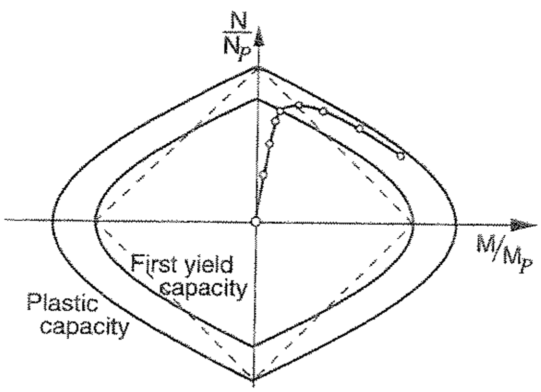


Figure 5-14 Plastic interaction surfaces

5.3.5 Column buckling

If proper considerations are taken in the FE modelling and element selection (ref. Section 5.3.2) then all formulations will be able to model column buckling. However, estimated buckling strengths are highly dependent on the modelling of the member. The theoretical formulations will reproduce the theoretical, 'ideal' behaviour of the member.

As a general rule, the software should be checked by simulating a single column. In most cases, some kind of calibration is required to produce buckling in accordance with any specific column curve.

For phenomenological models, this calibration is inherent in the formulations, since the column capacity, effective length factors and P-δ behaviour is explicitly given in the user input.

For general beam-column models, plastic hinge beam-column models and shell FE models, buckling is not introduced as result of a separate event or 'check' in the analysis. For these formulations the element stiffness is a function of the axial force in the element, and buckling takes place when the element stiffness is reduced to the extent that the column can no longer support additional loading. From this stage in the analysis, the axial force in the member has to be reduced in order to maintain equilibrium, and the column is said to have "buckled".

These element formulations are not tied to any column curves or capacity equations. Instead, the calibration can be achieved by introducing an artificial out-of-straightness on each member (ECCS, 1977). This is not a specification of any real or assumed geometric imperfection, but an equivalent measure to represent the combined effects of residual stresses and geometry imperfections found in real beam-columns (welding stresses, initial bow, section variations, thickness variations etc.).

The magnitude of this equivalent imperfection can be directly calculated from the column curve. The capacity of a pinned column with

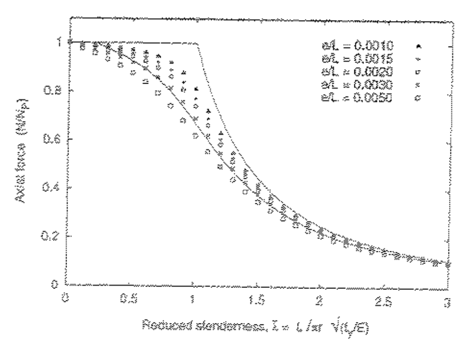


Figure 5-15 Column buckling estimates with different initial imperfections

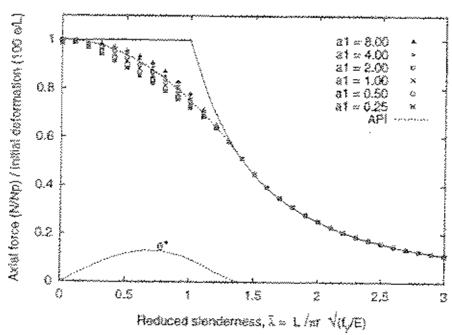
a geometric imperfection e subjected to axial force only, is given by

$$\frac{N}{P_{Cr}} + \frac{N \cdot e}{(1 - N/P_{Ek})M_U} \le 1 \qquad (5.1)$$

If failure of the column is to occur at a critical load P_{cr} (as defined by a specific column curve), then the parameter e can be solved as

$$e^* = (1 - \frac{P_{Cr}}{P_{Ek}}) \cdot (\frac{N_P}{P_{Cr}} - 1) \frac{M_U}{N_P} \qquad (5.2)$$

With this given out-of-straightness e^*, the column will fail when the axial force reaches the value P_{Cr}. This formulation can be used for any column curve; characteristic, lower percentile column curves or mean value column curves.



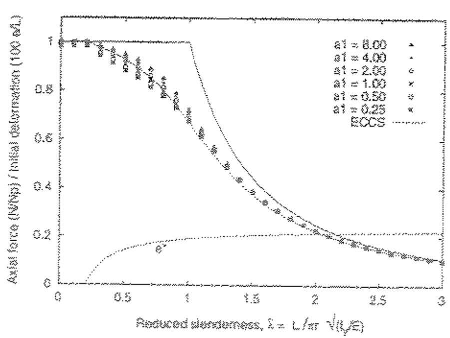


Figure 5-16 Equivalent initial imperfections for different column curves

Figure 5-15 illustrates the impact of initial imperfections on estimated column strength from non-linear analyses.

Figure 5-16 shows equivalent initial imperfections calculated from different column curves.

General beam-column models

Intermediate nodes along a member (ref. Section 5.3.2) should be offset according equation (5.2).

Plastic hinge beam-column models

Initial, stress-free deflections should be assigned according to equation (5.2).

Shell FE models

An initial imperfection pattern should be assigned according to equation (5.2).

All formulations

In the global analysis of the structure, each member should be assigned initial imperfections and plasticity parameters according to the member's slenderness and the selected column curve.

The imperfections should be assigned in a pattern sympathetic to the collapse mode of the structure. For fixed offshore structures this is

- in direction of the distributed loading on each member

or

- in direction of the global loading on the structure (in direction of the global base shear vector).

5.3.6 Beam-column behaviour

Effects of bending moment on buckling capacity and effects of axial force on bending capacity should generally be included in the structural analysis. For general beam-column formulations and plastic hinge beam-column formulations, this interaction between axial force and bending moment is automatically included in the formulation.

For some programs based on a phenomenological formulation, this effect must be modelled explicitly. This is especially relevant for members which are subjected to relatively higher bending moments, such as those closer to the water surface.

5.3.7 Local buckling

Detailed modelling of local buckling is not generally available in all non-linear formulations, but the impact on the member behaviour can still be captured to a reasonable extent. The most straightforward procedure is to substitute N_U for N_P and M_U for M_P according to the local buckling strength formulas. This captures the reduced section strength for those cases D/T-ratios where local buckling precedes the global column buckling.

Phenomenological models

For phenomenological formulations the increased load-shedding due to local buckling can be modelled by appropriate modification of the member's P-δ or M-θ curves. *General beam-column models*

In formulations where the cross-sectional characteristics are described by integrated P-δ and M-θ curves the reduction in section strength can be included by a reduction in the

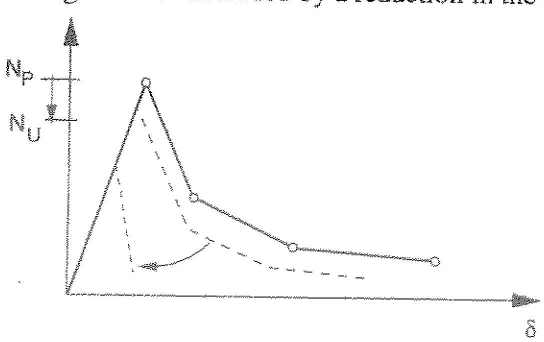


Figure 5-17 Modification of phenomenological member characteristics

P-δ or M-θ curves in the same manner as illustrated in Figure 5-17. The impact of this reduction will then automatically be included in the member behaviour.

In formulations where the cross-sectional behaviour is determined by integration points throughout the section local buckling may be included if the section shape changes when the strains exceed the critical values. However, such an option is not generally available.

Alternatively, the impact of local buckling can be modelled by a ductility limit on the σ–ε curves as illustrated in Figure 5-18. The ductility limit can set according to strain criteria for local buckling e.g. as given by Equation (4.5).

Plastic hinge beam-column models

Some beam-column programs include sophisticated formulations to detect local buckling and to model dent growth during the remaining loading. This is represented by a reduction in the yield surfaces as illustrated in Figure 5-19.

In other plastic hinge formulations, the effect of local buckling can be represented by a reduction in axial capacity and bending capacity, but the effect on the load-shedding characteristics of the member is not easily captured.

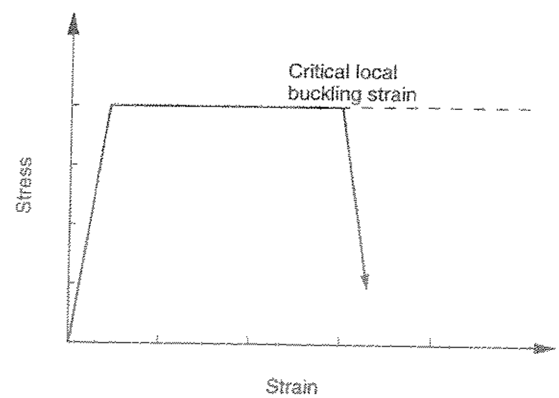


Figure 5-18 Modification of σ–ε curves to incorporate local buckling

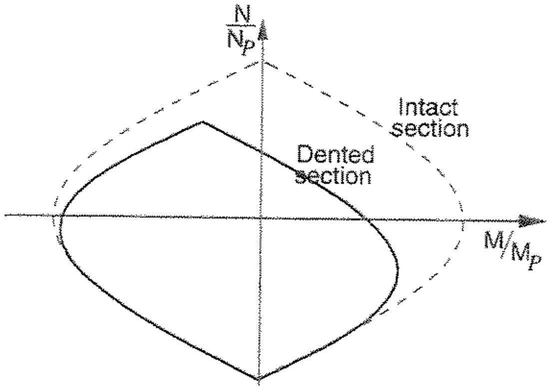


Figure 5-19 Plastic hinge modelling of local buckling

Shell FE models

A cross-section modelled by shell elements will generally not reproduce local buckling. However, to get a good representation of the buckle the FE model should be refined locally around the buckle.

The actual shape and direction of the buckle (inwards / outwards) is imperfection sensitive and is hard to predict even with a very detailed FE mesh. The shape of the buckle has a large influence on the predicted strains, but the 'global' P-δ (or M-θ) behaviour of the member should be reliably predicted, both pre- and post- local buckling.

5.3.8 Hydrostatic pressure

Models for hydrostatic pressure is not generally available in all non-linear formulations, but the impact on the member behaviour can be captured to a reasonable extent. The most straightforward procedure is to reduce the section capacities, i.e. substitute N_{hU} for N_P and M_{hU} for M_P according to the capacity equations for tubular members subjected to hydrostatic pressure as given e.g. in Section 4.2.5, equation (4.9). This should be accompanied by an increased load-shedding in the post-collapse range, as indicated for local buckling. Ref. Figure 4-10.

General beam-column models

The reduction in section strength due to the presence of hoop stresses is easily incorporated by specifying reduced section properties or reduced yield strength for the members in question. The effect of capped-end forces can however not be captured unless special load routines are incorporated.

Plastic hinge beam-column models

Some beam-column programs include formulations to account for hydrostatic pressure, including capped-end forces and reduction of section yield strength due to the presence of hoop stresses.

For other programs the effect of hoop stresses may be incorporated by specifying reduced section properties or reduced yield strength for the members in question. The effect of capped-end forces can not be captured unless special load routines are incorporated.

Shell FE models

A cross-section modelled by shell elements will represent the effect of hoop stresses accurately. But the effect of capped-end forces will still need special load routines to be captured.

5.3.9 Dented members

Detailed modelling of local buckling is not generally available in all non-linear formulations, but the impact on the member behaviour can be captured to a reasonable extent. The most straightforward procedure is to substitute N_U for N_P and M_U for M_P according to the capacity of the dented section as given e.g. in Section 4.2.5. This should be accompanied by an increased load-shedding in the post-collapse range, as done for local buckling.

Instead of subjecting the dented member to a separate unity check as prescribed by e.g. /4/, the actual out-of-straightness of the dented member should be modelled in the non-linear analyses, and the dented section modelled with reduced capacities.

General beam-column models

In formulations where the cross-sectional behaviour is determined by integration points throughout the section, the deformed section shape can be modelled directly. The effect of dent growth can however not be captured unless the section shape changes when the dented region is loaded.

Plastic hinge beam-column models

Some beam-column programs include sophisticated formulations to represent dented members and to model dent growth during the remaining loading. This is represented by a reduction in the yield surfaces as illustrated in Figure 5-19.

In other plastic hinge formulations, the effect of local buckling can be represented by a reduction in axial capacity and bending capacity, but the effect on the load-shedding characteristics of the member is not easily captured.

Shell FE models

A cross-section modelled by an appropriate mesh of shell elements will represent the damaged area and also account for further dent growth.

5.3.10 Member ductility

Ductility criteria formulated in terms of maximum strain are primarily applicable to shell FE formulations or general beam column formulations.

For general beam-column models strain criteria for local buckling (Section 4.2.4) and material ductility criteria (Section 4.2.7) can be directly applied. Strain criteria should be evaluated against the maximum von Mises strain or equivalent plastic strain computed from all strain components.

Shell element models normally produce reliable strain estimates until the occurrence of a local buckle. The direction of the buckle has a large influence on the predicted strains in the post-buckle regime. And since the direction of the buckle is hard to capture, the strain levels predicted for the buckled region may not be correct.

However, as long as the cross-section maintains its shape shell FE formulations will produce reliable strain estimates, e.g. to predict the onset of local buckling.

Beam-column formulations calculate deformations and rotations, and ductility limits in terms of strains are hard to apply.

For phenomenological modelling, the ductility limitations need to be included as a deformation limit in the P-δ curves or in the M-θ curves.

For plastic hinge beam-column models some simplified formulas may be used to estimate occurring strain levels. The obtained strain levels are highly dependent of the anticipated strain hardening and the acceptable strain level need to be seen in conjunction with the simplifications made in the material model. A bilinear simplification are proposed in /5/.

5.3.11 Cyclic degradation

Recent investigations /12/ indicate that compression members loaded only in the pre-buckling range will not be susceptible to cyclic degradation during extreme storm loading.

Thus, if the structure is not utilised beyond first member buckling, no further assessment of cyclic member behaviour should be required.

Otherwise, the effect of cyclic material degradation should be included in the analysis, and appropriate cyclic failure criteria applied.

Figure 5-20 illustrates typical cyclic material behaviour found for offshore steels. Some plastic hinge beam-column formulations include specialised formulations to capture cyclic material behaviour.

If such formulations are not available, an approximate representation can be achieved by specifying the fully degraded yield strength $f_{y,cyc} \approx 0.6 f_y$ for the entire loading range.

Failure is caused by through-thickness cracking in the most highly stressed area. This is captured by low cycle fatigue criteria, e.g. as defined by the extrapolated AWS curve (1983). In addition, an upper bound on the strains is defined by the monotonic local buckling criterion. See Figure 4-12.

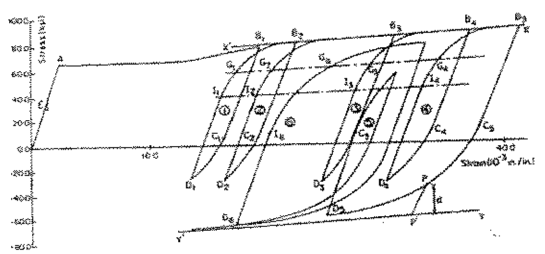


Figure 5-20 Cyclic material behaviour

5.4 Joint modelling

5.4.1 General

Tubular joint failure (in terms of limited capacity and/or gross deformations) should be accounted for, where appropriate, in ultimate strength analysis.

The applicability of the modelling approach chosen (see Section 5.4.5) should be reviewed in light of the significance of joint failure for the specific system response. It may be appropriate to conduct a sensitivity study to assess the influence of uncertainty in joint behaviour and modelling assumptions on the predictions.

The modification of member failure criteria to account for joint failure is not a satisfactory modelling device. It precludes the interaction between the loads, restraints and responses of the components.

5.4.2 Elastic joint flexibility

For typical structures the joints may be modelled as rigid connections at the brace / chord intersection. For conventional structures this introduces some conservatism in the analysis results.

For large diameter chords or short braces, local joint stiffness should be considered.

Joint flexibility may be modelled by separate finite elements introduced between a node at the chord/brace intersection (chord surface) and the chord centre node. The flexibility properties can be assigned according to formulae developed by various researchers. See /19/, /20/, /21/ and /22/

5.4.3 Joint eccentricities

In ultimate strength analysis to determine global collapse, modelling of eccentricities need be no more complex than for conventional elastic analysis practice i.e. eccentricities < D/4 need not be modelled explicitly and the intersection of brace and chord centrelines can be modelled as concentric at a single node point.

5.4.4 Screening

The approach to joint modelling may depend on software facilities. In some cases joint modelling options may be activated throughout the structure irrespective of utilisation level. In other cases a first pass screening may be performed to confine the complexity of joint failure 'modelling' to critical areas. A suitable procedure is shown in Figure 5-21 beginning with a model assuming 'rigid' joints:

1. perform ultimate strength analysis;

2. evaluate the utilisation of tubular joint intersections at maximum load;
3. screen out 'rigid' joint assumptions for those joints where non-linear deformations or limiting capacity may influence the response in Step 1 (guidance on screening criteria is given in the Commentary);
4. update model to include representation of 'failure' for screened joints.

Repeat until all joints where 'failure' potentially influences the system response have been included (generally one cycle is sufficient).

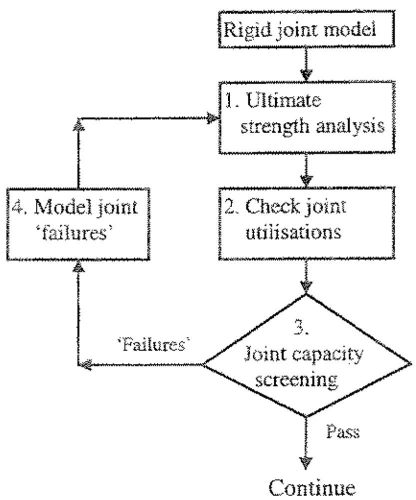


Figure 5-21 Screening procedure for modelling non-linear joint characteristics

5.4.5 Joint representation

The representation of joint 'failure' in ultimate strength analysis depends on the software facilities. Potential approaches, with increasing degree of sophistication illustrated in Figure 5-22, are:

1. rigid-plastic assumption with joint flexibility neglected, limit load based on joint capacity formulae and post-peak response characteristics neglected;
2. parametric representation of the increasing flexibility up to peak load and possible inclusion of post-peak load shedding/hardening;
3. explicit inclusion of FE model comprising non-linear shell elements.

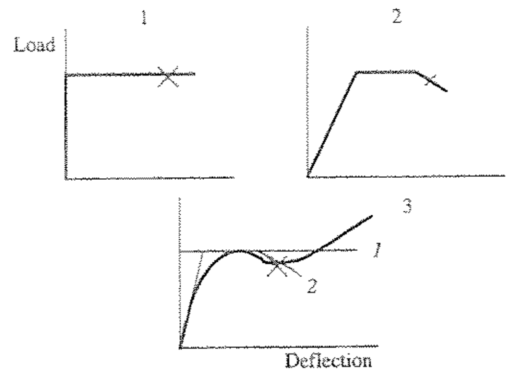


Figure 5-22 Approaches to joint modelling including ductility limits (x)

The selection is determined by the available software and consideration should be given to the advantages and limitations of each method.

Figure 5-23 illustrates the introduction of joint 'elements' or 'springs'. Figure 5-23 illustrates the introduction of separate joint 'elements' or 'springs' to represent joint behaviour.

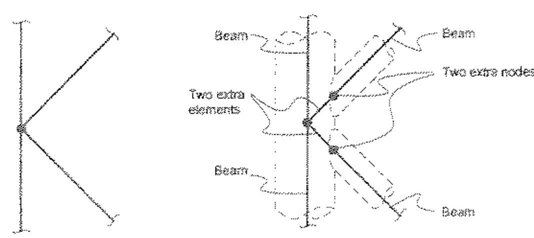


Figure 5-23 Introduction of additional elements for joint modelling

A simplifying approximation may be to disconnect the brace element from a node where joint failure is anticipated. In many instances this will give a lower bound to system capacity. In some cases the significant redistribution of load effects may precipitate earlier failure in other components undermining systems strength. If a disconnect strategy is adopted, appropriate validity checks should be carried out.

5.4.6 Capacity modelling

If ultimate strength analysis is being performed to obtain a 'best estimate' of system strength, it is important to ensure that all component response criteria are defined on a consistent (ie mean) basis. Any combination of mean and characteristic values would be inconsistent and would give misleading results potentially affecting the predicted mode of failure and system capacity.

The relation between mean and characteristic joint capacities in relation to underlying data is shown schematically in Figure 5-24.

Use of lower fractile characteristics would give a more consistent component reliability accounting for variability across different component types and response modes.

Joint capacity can be assigned on the basis of parametric formulae of the form:

$$P_u = \frac{F_y T^2}{\sin\theta} f(geometry\,chordload) \tag{5.3}$$

$$M_u = \frac{F_y T^2 d}{\sin\theta} f(geometry\,chordload)$$

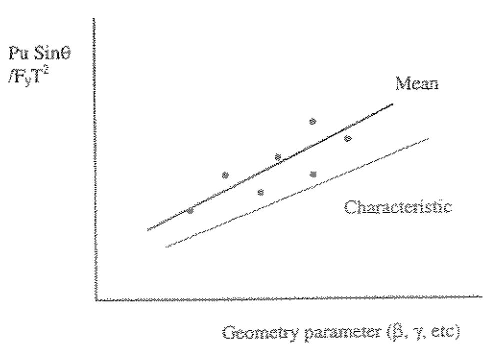


Figure 5-24 Test data, mean and characteristic capacities

Characteristic and lower bound formulations of this form can be found in codes of practice (eg API RP 2A). The Draft ISO code /4/ and information in the Commentary provide appropriate function data for a mean capacity representation for simple joint geometries and loading modes.

The potential for force-moment interaction should be checked and allowed for, where appropriate.

Similarly the effect of co-acting chord stresses on joint capacity, particularly for X configurations, should be accounted for.

5.4.7 Flexibility modelling

The initial flexibility of tubular joints is very small in comparison with the softening associated with joints approaching failure and can generally be neglected in ultimate strength analysis. Where joints become loaded at or near their capacity however, the large deflections and rotations should be accounted for.

The deflections and rotations corresponding to a specific non-dimensionalised load level can be expressed in the form:

$$\frac{\delta}{D} = \frac{P}{P_u} f(geometry) \tag{5.4}$$

$$\phi = \frac{M}{M_u} f(geometry)$$

Several JIPs have developed formulae by joint type and loading mode with reference to isolated joint test data but these remain confidential to sponsors. As there is no published source of P-δ, M-ϕ response characteristics for joints at, around and beyond peak capacity, a set of piece-wise linear formulations is given in the commentary which may be used.

5.4.8 Joint modelling - Caution

The complexity and diversity of joint types and failure modes, precludes anything other than a phenomenological representation of test data. However, isolated test data are by necessity simplistic in terms of the load combinations, interactions and boundary constraints imposed when compared with conditions experienced by a joint in a frame. This needs to be recognised and care taken when applying these data and/or formulations based on them.

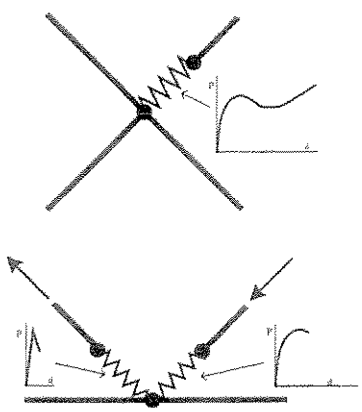


Figure 5-25 Phenomenological spring representation of X and K joint characteristics

5.4.9 Cyclic degradation

Failure is caused by through-thickness cracking in the most highly stressed area. This may be addressed with reference to low cycle fatigue criteria, e.g. as defined by the extrapolated AWS curve (1983). Alternatively, large plastic deformations may lead to incremental collapse. Indicative limits on the sustainable deformations at joints are given in the commentary.

5.5 Foundation modelling

5.5.1 Soil capacity formulation

Numerous soil capacity formulations have been published, based on empirical data, soil mechanics or both. Some are restricted to one spesific soil type (cohesive / cohesionless soil) whereas other are general. Choice of formualtion should be based on several criteria:

- Accuracy: that the formulation has been sufficiently validated for the soil type in question.
- Complexity: that the model is based on a limited number of parameters, and that the parameters are easy to determine from standard sampling methods.
- Familiarity: that the formulation and the input parameters are reasonably familiar to the professional engineer, such that the chance of human errors and mis-interpretation of the method is limited.

Recent evaluation of pile soil formulations /8/ indicate that most modern geotechnics formulations give reasonable accuracy, considering the inherent uncertainty in the soil. The chance of mis-interpretation of data is considered more onerous than the inaccuracies inherent in the formulations themselves. Based on this, the formulation presented in API RP2A 20th edition /2/ is recommended.

5.5.2 Use of cyclic degraded backbone curves

As discussed in Section 4.5.4, the *p-y* and *t-z* curves presented in most codes are not load-deformation curves for non-linear soil behaviour, but represent the fully degraded soil resistance for cyclic loading at given deformation levels.

Therefore, it is not strictly correct to use the *p-y* and *t-z* curves directly as P-δ curves in a non-linear analysis. However, using the *p-y* and *t-z* curves as P-δ curves is considered to give results on the safe side, and is consistent with common procedures in ordinary, 'linear' ultimate limit state analyses.

5.5.3 Cyclic versus monotonic capacity

At a certain depth along a pile, the soil will no longer experience cyclic (fully reversed) loading. In an extreme storm situation, the reverse loading (wave trough) is typically some 30% of the incoming load (wave crest), when the effect of in-line current is included, see Section 6.8.

Applying a load history as indicated in Section 6.8, it is possible to estimate the depth at which the soil loading ceases to be cyclic. From this depth and downwards, it is reasonable to assign soil properties for monotonic loading.

5.5.4 Loading rate effects

In the event that rate effects are to be included, any increase in capacity due to rate effects should only be assigned to the dynamic part of the soil response. See Section 4.5.5.

6 Load Modelling

6.1 General

Loads must be modelled in sufficient detail to ensure that the relevant failure modes are activated in non-linear analysis program

This section gives a set of modelling recommendations to help make the non-linear analysis tool produce reliable results that conform to recognised failure criteria and design formulations.

This section describes the appropriate representation of loads for non-linear analysis methods. The actual analysis execution is treated in Chapter 7.

6.2 Wave kinematics and wave loads

Wave kinematics and wave load models should be taken as for conventional ultimate limit state analyses.

6.3 Distributed member loads

The presence of lateral member forces reduces the member's buckling strength. Thus, environmental wave and current forces should as far as possible be modelled by distributed member loads.

6.4 Load combinations

The principle of load superposition is invalid for non-linear analyses. Instead, the primary load cases must be superimposed before they are applied to the structure.

6.5 Load application

6.5.1 General

In a non-linear pushover analysis, the loads are applied in sequence. Time invariant dead and live loads are first incremented up to their factored value. The environmental loads for the loading direction in question are then increased gradually until collapse of the platform. The RSR ("Reserve Strength Ratio") is defined as the ratio between the base shear at platform collapse and that of the 100 year environmental load.

Several procedures may be used to apply the environmental loading.

6.5.2 100-year environmental load vector

One common procedure is to calculate the 100-year load vector, and increment this load vector proportionally until collapse. This load vector includes wave, wind and current forces with a notional return period of 100-years, considering probability of simultaneous occurrence. See Figure 6-1. This implies that the wave height is fixed (at the 100-year wave height), and the analysis determines how many *multiples of this load* the structure can take before it collapses

This load application procedure does not account for changes in load pattern as the wave height is increased. As long as no additional horizontal elevations are submerged, the load incrementation pushover procedure gives reliable results.

6.5.3 Wave height incrementation

To capture wave-in-deck forces, the wave height itself should be incremented, and the load vector changed according to the actual position of the wave crest in each case. This is illustrated in Figure 6-2.

The wave and current loading should then be calculated at different stages during the load history, and the applied load vector adjusted as higher elevations of the structure is submerged.

6.5.4 10 000-year load vector

A simpler way to include deck inundation and wave-in-deck loads are simply to calculate the 10 000-year load, and increment this load vector until collapse. This load vector will then include loads caused by deck inundation and sea loads on members located above the 100-year wave crest.

6.6 Deck inundation

6.6.1 General

A check on deck inundation is recommended if the analysis is based on incrementation of the 100-year load vector.

If the wave height ($h_{collapse}$) associated with the calculated collapse capacity is large enough to contact the deck, then the wave height incrementation method is recommended.

6.6.2 Load factor vs. wave height

Using a power relation between base shear and wave height

$$Q_{BASE} = C \cdot h^{\alpha} \tag{6.1}$$

the following relation is obtained between wave height and the Environmental Load Multiplier (ELM). ELM is the ratio of the applied environmental loads to the characteristic environmental loads.

$$\frac{h_{collapse}}{h_{100}} = \left(\frac{Q_{collapse}}{Q_{100}}\right)^{1/\alpha} = ELM^{1/\alpha} \tag{6.2}$$

A factor α=2.0 would correspond directly to the drag term in Morrison's equation The actual α-value may be determined on a case-by-case basis.

The wave height corresponding to the required capacity can then be compared to the air gap of the structure, ref Section 6.8. The asymmetry of the wave should be taken into account, e.g. the crest height of a Stokes wave is typically some 60-65% of the total wave height.

6.6.3 Wave-in-deck forces

Wave-in-deck forces may be calculated according to API or other recognised formulations.

A simple way of including such forces in the analysis is to introduce additional load-attracting in the cellar deck level of the structure. The members should be positioned to take forces as soon as the wave crest inundates the bottom part of the cellar deck. In some load generation programs, members start attracting loads once the wave crest passes the centroid of the member. For such programs the members should be located at cellar deck - ½ the depth of the bottom deck beam. The height of the members, the C_D, or the wave kinematics model should be adjusted in such a way that the required wave forces are introduced in the structure.

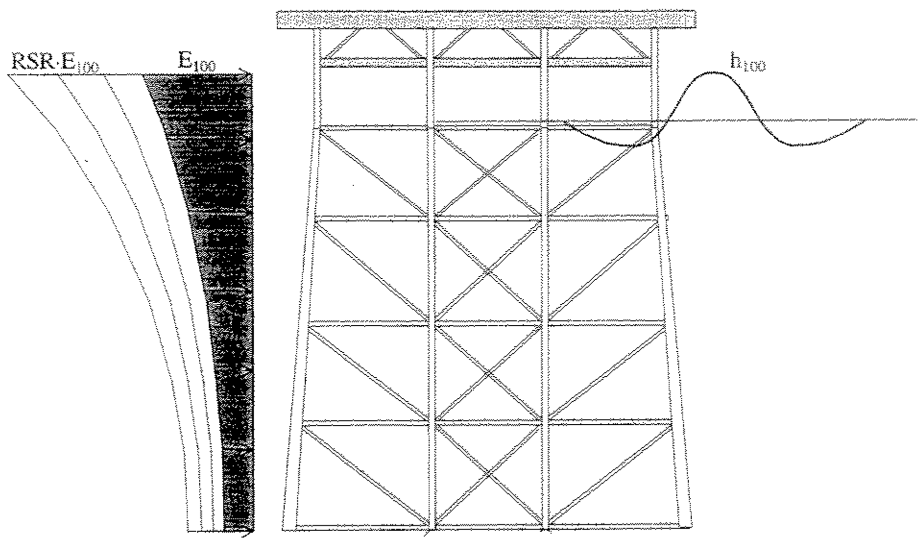


Figure 6-1 Incrementation of a fixed, reference load vector, E_{100}

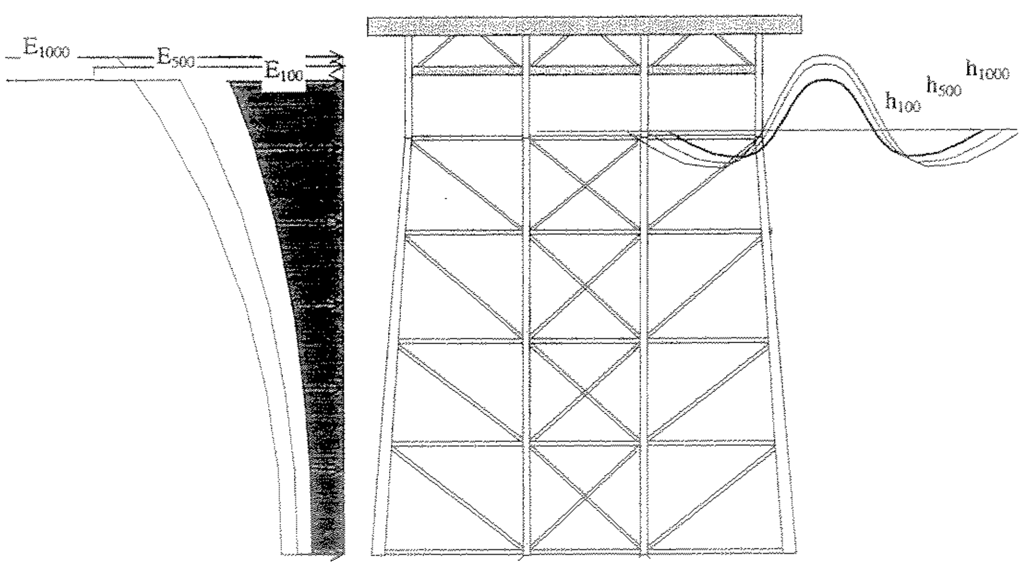


Figure 6-2 Incrementation of wave height; load vector calculated according to actual position of the wave crest, considering wave-in-deck forces.

6.7 Cyclic storm loading

In case a separate check of cyclic capacity is necessary (see section 4.4.2), a representative load history cyclic storm loading may be derived as follows.

The loading on offshore structures is characterized by wave, wind and current forces from all geographic directions. In a storm situation, the sea elevation is approximated by a narrow-band Gaussian process. The distribution of individual waves is then assumed Rayleigh distributed:

$$F_h(h) = 1 - \exp\{-\frac{1}{2}\left(\frac{h}{\sigma}\right)^2\} \qquad (6.3)$$

The ratio between the highest wave, second highest, third highest waves is then given by

$$\frac{h_m}{h} = \sqrt{1 - \frac{\ln m}{\ln N_P}} \qquad (6.4)$$

where m denotes the m'th largest wave, and N_P is the number of waves in a storm, typically 1000 – 2000. Using Equation (6.2) as the relationship between wave height and base shear, the distribution of the m highest loads in an extreme storm situation is given by

$$\frac{Q_m}{Q_{\max}} = \left(1 - \frac{\ln m}{\ln N_P}\right)^{\alpha/2} \qquad (6.5)$$

where α is discussed in section 6.6.2.

A representative load history for cyclic extreme storm loading is given by /11/. Time invariant dead and live loads are first incremented up to their factored value. It is then to be demonstrated that the structure shakes down to an elastic state when exposed to the following load cases in sequence;

1. multiple cycles of the unfactored 100-year environmental load E_{100} from primary and opposing direction
2. an ordered load sequence representing the 10 000 year (10^{-4}) extreme storm loading $E_{10\,000}$ or, alternatively, the factored ($\gamma_m \cdot \gamma_f$) 100-year environmental load E_{100}.
3. multiple cycles of the unfactored 100-year environmental load E_{100} from primary and opposing direction

The waves in the 10 000 year storm sequence are given by (6.5). The 100-year load cycles ensure that low cycle fatigue does not occur in damaged members at relatively low storm intensities, and that the structure is stable after the passage of the extreme event.

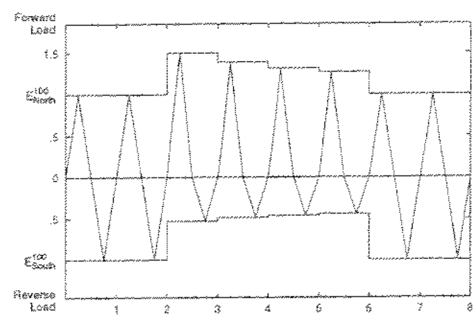


Figure 6-3 Extreme storm cyclic loading

The 10 000 year storm waves are applied in decreasing sequence. The forward loading typically comprise forward wave + forward current + forward wind. The reverse loading typically comprise reverse wave (wave trough) + forward current and no wind /12/.

6.8 Determination of crest height

The wave crest may be defined as the vertical distance between the peak surface elevation within a wave cycle and the Mean Water Level (MWL) see Figure 6-4.

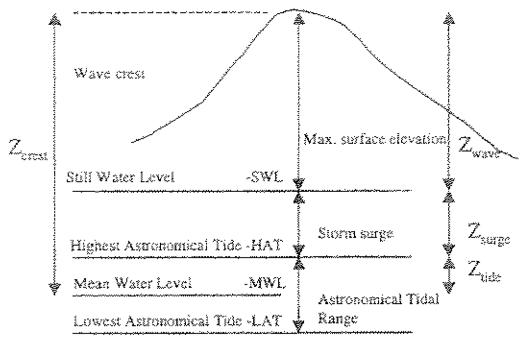


Figure 6-4 Definition of maximum wave crest

The wave crest Z_{crest} accounts for the the cumulative effects of waves, surge and tides, and can be expressed as the sum:

$$Z_{crest} = Z_{wave} + Z_{surge} + Z_{tide} \qquad (6.6)$$

All three effects contributing to the wave crest are associated with randomness, but the main source is the wave height. The determination of the wave crest may be based on wave height statistics as long as a recognised wave model (e.g. Stokes 5^{th}) is used to calculate the resulting wave force. However, to check the possibility of deck inundation, the crest height should be determined from crest height statistics, since the Stoke 5^{th} wave do not describe the wave profile irregularity properly.

7 Execution of Ultimate Strength Analyses

7.1 General

Non-linear analysis of jacket structures demands a methodical approach and a number of systematic checks are appropriate to verify the robustness of ultimate system strength predications.

7.2 Preparation

Data from previous analyses of the jacket should be assembled. Elastic code check analyses can be instructive, helping direct the modelling effort and providing a reference base for verifying initial non-linear responses at component and system levels. For example:

- By establishing the direction of moments acting in member, initial imperfections can be introduced in a detrimental sense.
- If the utilisations of tubular joints are significantly less than for members, it may be assumed that their non-linear response characteristics will not influence overall system behaviour and the complexity of joint modelling need not be introduced initially.

Where elastic (code checked) results are not available, consideration should be given to performing an initial elastic analysis. It provides a sound basis from which the non-linear analyses can be developed and may bring overall efficiencies in the assessment process.

However, the limitations of elastic analysis should also be recognised. Once plasticity develops, component responses soften and loads redistribute, the correspondence with the elastic force distribution will diminish. The approach therefore provides only an initial basis for validation and the potential for other components to contribute to the collapse scenario needs to be re-examined throughout the non-linear analysis.

7.3 Execution strategy

To facilitate the interpretation and verification of ultimate strength predictions, a systematic approach to the non-linear analysis is recommended. For each scenario a sequence of analyses of increasing complexity and increasing realism should be performed beginning with the structure alone then introducing, for example, soil-structure interaction and non-linear joint characteristics in turn, as appropriate. At each stage the results should be examined and the influence of the modelling changes examined (see Section 7.7).

7.4 Considerations for some failure modes

7.4.1 Member yielding

If yield hinges are formed in members it is necessary to ensure that the cross-section is stable under the induced rotations see 4.2.4. The presence of hydrostatic pressure will for tubular members reduce the ability of the cross-section to sustain the full moment capacity under increased rotations. See 4.2.6.

7 Execution of Ultimate Strength Analyses

7.1 General

Non-linear analysis of jacket structures demands a methodical approach and a number of systematic checks are appropriate to verify the robustness of ultimate system strength predications.

7.2 Preparation

Data from previous analyses of the jacket should be assembled. Elastic code check analyses can be instructive, helping direct the modelling effort and providing a reference base for verifying initial non-linear responses at component and system levels. For example:

- By establishing the direction of moments acting in member, initial imperfections can be introduced in a detrimental sense.
- If the utilisations of tubular joints are significantly less than for members, it may be assumed that their non-linear response characteristics will not influence overall system behaviour and the complexity of joint modelling need not be introduced initially.

Where elastic (code checked) results are not available, consideration should be given to performing an initial elastic analysis. It provides a sound basis from which the non-linear analyses can be developed and may bring overall efficiencies in the assessment process.

However, the limitations of elastic analysis should also be recognised. Once plasticity develops, component responses soften and loads redistribute, the correspondence with the elastic force distribution will diminish. The approach therefore provides only an initial basis for validation and the potential for other components to contribute to the collapse scenario needs to be re-examined throughout the non-linear analysis.

7.3 Execution strategy

To facilitate the interpretation and verification of ultimate strength predictions, a systematic approach to the non-linear analysis is recommended. For each scenario a sequence of analyses of increasing complexity and increasing realism should be performed beginning with the structure alone then introducing, for example, soil-structure interaction and non-linear joint characteristics in turn, as appropriate. At each stage the results should be examined and the influence of the modelling changes examined (see Section 7.7).

7.4 Considerations for some failure modes

7.4.1 Member yielding

If yield hinges are formed in members it is necessary to ensure that the cross-section is stable under the induced rotations see 4.2.4. The presence of hydrostatic pressure will for tubular members reduce the ability of the cross-section to sustain the full moment capacity under increased rotations. See 4.2.6.

If the capacity under large rotations is uncertain it should be neglected in the calculations.

For members subjected to repeated loading e.g. waves, it is necessary to check that repeated yielding do not lead to failure. See 4.2.8. For the main elements in traditional jacket structures gross sectional yielding of members exposed to wave loads do not normally lead to cyclic failure. The reason is that in order to have any form of cyclic degradation, the minimum condition is that the same locations of a structure must yield both under forward and reverse loading. See Commentary to 4.4.2.

Normally the maximum ultimate load will occur for small strains compared to the rupture strain. But in special cases it may be necessary to check that the plastic strains do not exceed critical strains. See 4.2.7.

7.4.2 Local buckling

Both the cross-sectional capacity as well as the post ultimate behaviour is dependent upon local buckling, see 4.2.4. For tubular members the stability of the cross-section will also depend on the presence of hydrostatic pressure. See 4.2.6. If local buckling will take place the cross section should be assumed ineffective for further loading, unless reliable information of the post ultimate behaviour is available.

7.4.3 Member buckling

The analysis must be carried out in a way such that fabrication tolerances and member residual stresses are accounted for See 5.3.5. Assessment of cyclic member behaviour is only required if the structure is utilised beyond first member buckling. See Commentary to 4.4.2.

If the structure is utilised beyond first member buckling, but the member does not yield under the reverse (wave trough) loading, then no low-cycle fatigue will take place, and no further assessment of cyclic member behaviour is required.

7.4.4 Joint failure

In cases where the joints are significantly stronger than the plastic bending capacity of the members the joint capacity and behaviour need not be modelled.

If the capacity of the joints is similar to or weaker than that of the corresponding members the behaviour of the joints and their ductility limits needs to be modelled and checked. See4.3.

When joints are loaded beyond their proportional limit and exposed to cyclic loading, it is necessary to carry out a check that no failure occur do to repeated yielding. See 4.3.10. The proportional limit for tubular joints is generally difficult to define. In lieu of relevant data it is proposed to use the limit for elastic behaviour in the idealised proposed joint flexibility diagram in the Commentary.

If the joint is utilised beyond the proportional limit, but the joint does not yield under the reverse (wave trough) loading, then no low-cycle fatigue will reduce the ultimate capacity, and no further assessment of cyclic member behaviour is required.

7.4.5 Other limit states

In exploiting ultimate system strength, care must be taken that other limit states are not violated e.g. global deflections exceeding safety or serviceability criteria for e.g. equipment or risers.

7.5 Load application

The principle of load superposition is not valid for non-linear analysis; the sequence and location of non-linear events is dependent on the pattern of load application.

For offshore jacket structures, it is appropriate to apply constant topside and self weight loads, followed by environmental loads (including wind, waves, currents and inertial actions). The incrementation of environmental loads depends on the purpose of the analysis and basis of interpretation (Chapters 3.4, 6.5 and 9):

7.6 Solution control

Automatic solution controls are generally available but manual intervention may be required. Sudden changes in stiffness, rapid unloading and alternative equilibrium conditions in the non-linear region can give numerical instability and present challenging problems for all analysis software; small step sizes may be required to coax the solution.

Apparent solution difficulties should be investigated in terms of the non-linear events being predicted. Failure of an analysis does not necessarily mean that the ultimate strength of the system has been reached. Solution should be continued until a clearly defined peak or sustained limit load has been reached and the post-ultimate response characteristics determined.

7.7 Verification

In conventional design procedures, the structural performance is described by unity checks for the most heavily utilised components. For ultimate strength analysis, the structural performance is described by the global collapse mechanism, the system capacity and the sequence of non-linear events.

The response is often complex with multiple interactions influenced by the modelling and analysis strategy adopted. The primary and essential validation of non-linear analysis results comes from understanding the development of the global collapse mechanism. The following minimum checks are recommended:

- Is collapse behaviour consistent with bracing system / apparent redundancy?
- Has a clear failure mechanism developed?
- Can global P-Δ behaviour be explained by non-linear events at the local level (e.g. component capacities, load shedding, redistribution, etc)?
- Are non-linear component responses credible? (Reference failure modes described in Chapter 4).
- Is (initial) sequence of non-linear events (yielding, buckling, joint failure etc) consistent with available elastic code check results?
- Is the calculated mechanism valid? eg. is plastic deformation of components within ductility limits (if these are not included in the element formulation)?
- Have all relevant non-linear component responses been represented?
- Do all simplifying assumptions made at the outset remain valid in light of the response determined (eg. if large deflections have been calculated, can lateral load resistances generated from the inclination of springs at releases be shown to be insignificant?)

Further guidelines are given in Chapter 10.

7.8 Sensitivity analyses

In performing ultimate strength analysis deterministic values are necessarily adopted for physical properties and component responses despite inherent randomness and model uncertainties. Actual values deviating from these assumptions may result in different component capacities or response characteristics. These in turn might alter the sequence of non-linear events and load redistribution defining the collapse scenario, ultimate system capacity and post-peak behaviour. Depending on the purpose of the analysis and intended use of the results, it will generally be appropriate to test the robustness of the calculated response. Appropriate sensitivity analyses should be defined on a case by case basis with reference to known uncertainties and the sequence and characteristic of non-linear events within the structure.

In general it should be recognised that 'brittle' response characteristics which precipitate rapid load shedding (e.g. buckling) may be particularly sensitive to uncertainties whereas 'ductile' behaviour (e.g. gradual yielding) may be more robust.

7.9 Dynamic loading effects

7.9.1 General

Fixed offshore structures with lowest natural period less than 3 seconds are normally treated as statically loaded and consequently dynamic effects may be neglected in the non-linear analyses. For structures with longer natural periods dynamic effects need to be considered.

The impact on the ultimate capacity from dynamic effects will be different for a structure where the non-linear analysis reveals a ductile development of the structural collapse compared with a more brittle mode. See Figure 7-1.

7.9.2 Recommendations for brittle structures

The dynamic collapse capacity for brittle structures should be obtained by either of the following methods:

- dividing the static pushover capacity by a dynamic amplification factor (DAF)
- including an equivalent inertia load case in the load history

The dynamic amplification factor need to be established in line with ordinary procedures for dynamic analyses see e.g. API RP2A /2/ or /3/.

7.9.3 Recommendation for ductile structures

The dynamic collapse capacity for ductile structures can for simplicity be taken as the static pushover capacity without use of a knock-down factor due to elastic dynamic action. Separate investigations indicate that the real dynamic capacity may be of the order 10-20% higher than the static capacity /14/.

Semi-ductile structures should be treated as brittle unless higher dynamic capacity can be justified by separate studies.

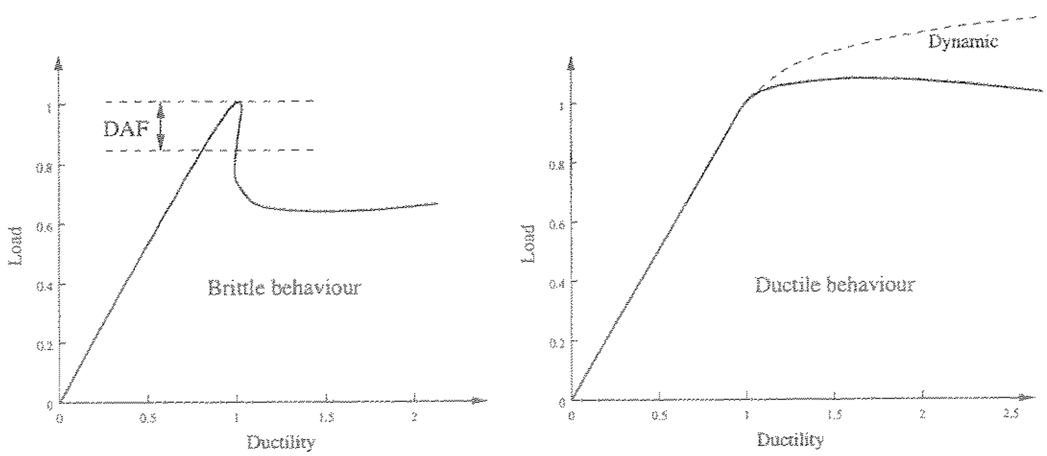


Figure 7-1 Dynamic loading effects for brittle and ductile structures

7.9.4 Time-domain dynamic collapse analyses for ductile structures

For time domain analyses, the start-up condition prior to the extreme wave is important. It is recommended to define a load history comprising at least three wave periods, and linearly increase the load over the first two cycles to provide a start-up condition for the extreme wave.

The force history should be normalised such that the peak value corresponds to the static collapse load. In the subsequent dynamic analyses, the load history should be scaled up (or down) and stepped through the structure at successively larger (or smaller) intensities.

The failure criterion for the analyses should be defined by the amount of deformation that can be tolerated without degradation of structural capacity or to ensure that the platform remains operable.

7.10 Precautions in non-linear analysis

The principles of non-linear behaviour must be properly accounted for in the analysis, and some differences between linear and non-linear analyses should be observed:

- Linear superposition is not valid and response may exhibit a load-history dependence
- Modelling with non-linear "hyper-elastic" or "elastic restoring" springs should be done with caution. Even if the overall structural behaviour is a monotonic increase in load and deformations, re-distribution of internal forces may lead to local unloading in some regions of the structure. Unloading of a "hyper-elastic" spring will then lead to spurious energy being *fed back into the structure*, e.g forcing the ends of the spring apart.
- Member releases should be used with caution. The orientation (local axes) of the member will change during the non-linear analysis, generating spurious loads at member releases. This modelling should be used only if the spurious loads and resistance at the member releases can be shown to be insignificant.

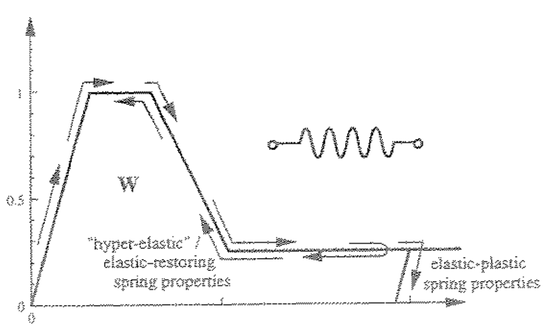


Figure 7-2 Unloading of "hyper-elastic" springs

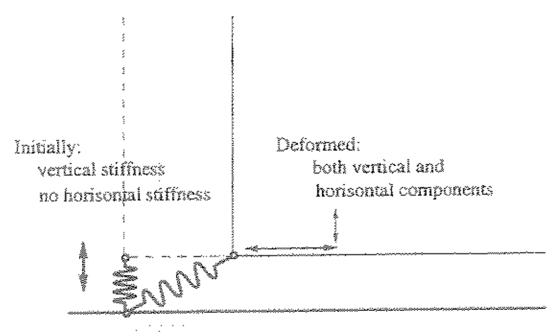
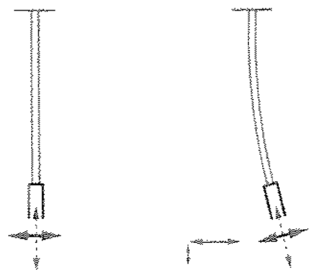


Figure 7-3 Change of member orientation during non-linear analyses



Axial deformations released

Initially:

transfer of lateral forces
no vertical forces

Deformed:

both vertical and
horisontal force components

Figure 7-4 Spurious loads in member releases

8 Structural Reliability Issues

8.1 General

A formal assessment of structural reliability needs to include considerations of the statistical variation in both loads and resistance. Most codes introduce in one way or another a factor of safety. Today most codes do not provide detailed requirements to how analyses made with non-linear methods shall be executed to obtain an adequate safety level. In this chapter, this topic is discussed.

8.2 Linear and non-linear analysis

In general, all structures behave non-linearly when loaded close to their ultimate capacity. Nevertheless, the determination of the internal forces and moments in a structure is in nearly all offshore engineering practice done by linear elastic analyses. Then the failure criterion is usually yielding of outer fibre in the most loaded cross-section, or buckling of any individual member as an isolated beam column. This failure criterion is established as a matter of convenience since the elastic analysis cease to represent the actual structural behaviour at this level of loading.

The yielding or buckling of a member is often referred to as component failure. This is not a failure in a physical sense, but rather violation of the selected failure criteria.

Structures designed according to linear methods will, due to this failure definition, in most cases exhibit a larger margin of safety than the safety factors should imply. The quantity of this margin will vary between different type of structures and load conditions and from one failure mode to another.

8.3 LRFD format

8.3.1 General

The load and resistance factor design format (LRFD) is the design format which presently is the governing format for new structural design codes. An example is the new ISO 13819 Part1 for Offshore structures /1/. In this code the term limit state is defined as follows:

"The structural performance of a whole structure or part of it shall be described with reference to a specified set of limit states beyond which the structure no longer satisfies the design requirements."

In the ISO 13819 /1/ four categories of limit states are defined. The category of the so-called ultimate limit state (ULS) are those which correspond to the maximum resistance to applied actions. In most cases, the maximum resistance is reached when the structure is loaded beyond its elastic limits.

This implies that non-linear analyses may be employed directly for checking of ultimate limit states for this design format.

The limit state design codes will require additional checks when the analysis assumes the structure to be loaded in excess of the elastic limit. For instance, checks to avoid low cycle fatigue from repeated yielding will be required, see e.g. NPD Guidlines /9/.

The limit state design format makes use of partial safety factors and characteristic values for load and resistance to ascertain the required safety. See Figure 8-1.

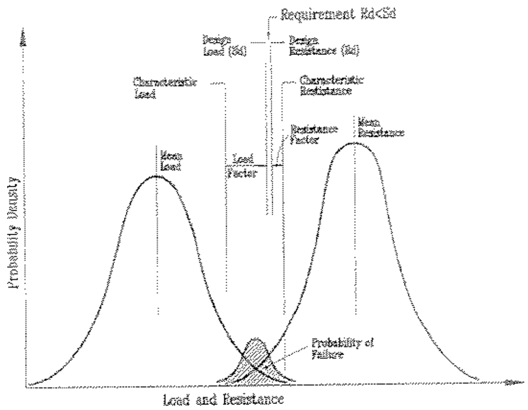


Figure 8-1 Limit state design format

The requirement is that the design resistance should be greater than the design action (load) or expressed as:

$$R_d \geq S_d \qquad (8.1)$$

Where:

$R_d = R_k\,\varphi$	Design resistance (API)
$R_d = R_k/\gamma_m$	Design resistance (NPD)
$S_d = S_k\,\gamma_f$	Design action
$R_k =$	Characteristic resistance
$S_k =$	Characteristic action
$\varphi =$	Resistance factor (API)
$\gamma_m =$	Partial material factor (NPD)
$\gamma_f =$	Partial action factor

Figure 8-1 shows that it is equally important to the safety how the characteristic values are defined as the size of the safety factors.

To apply this requirement to a structure, which is assessed by non-linear analysis, it is recommended to follow one of the following procedures, denoted Method A and Method B. Method B is generally recommended.

8.3.2 Method A

In Method A the behaviour and the resistance of the components and members of the structure is adjusted in the analysis to represent the characteristic resistance divided by the appropriate material factor. The procedure to be followed will be:

1) Use characteristic strength (e.g. f_y) multiplied with resistance factor φ or divided by material factor γ_m as input.
2) Apply non-environmental actions (e.g. dead loads) until their factored value.
3) Increment the characteristic environmental actions by applying an Environmental Load Multiplier (ELM) until the structure has reached its ultimate capacity.
4) Compare the value of the ELM with the required action factor for environmental actions.

For a safety format according to API LRFD /3/ the inequality (8.1) may then be written as:

$$R(\varphi_{Mem} R_{Mem}, \varphi_{Jnt} R_{Jnt}, \varphi_{Soil} R_{Soil}) \geq \gamma_D P + \gamma_L L + \gamma_E E \qquad (8.2)$$

For a safety format as used by NPD /9/ equation (8.1) may then be written as:

$$R\left(\frac{R_{Mem}}{\gamma_{Mem}}, \frac{R_{Jnt}}{\gamma_{Jnt}}, \frac{R_{Soil}}{\gamma_{Soil}}\right) \geq \gamma_D P + \gamma_L L + \gamma_E E \qquad (8.3)$$

where:

R	=	Ultimate platform resistance
R_{Mem}	=	Member resistance
R_{Jnt}	=	Joint resistance
R_{soil}	=	Soil resistance

P = Permanent loads
L = Live loads
E = Environmental load
φ_{Mem} = Resistance factor for member
φ_{Jnt} = Resistance factor for joint
φ_{Soil} = Resistance factor for soil
γ_{Mem} = Material factor for members
γ_{Jnt} = Material factor for joints
γ_{Soil} = Material factor for soil
γ_D = Action factor on dead load
γ_L = Action factor on live loads
γ_E = Action factor on environmental loads

The ratio of the maximum capacity in the analysis to the characteristic environmental loads (ELM) should then be compared to the action factor for environmental loads γ_e. In general both load combinations (ordinary and extreme in API or load combination a and b in NPD) need to be checked.

When factoring the characteristic resistance, care should be taken that the material factors only affect the *strength* of the component, and do not affect the *stiffness*. This is illustrated in Figure 8-2.

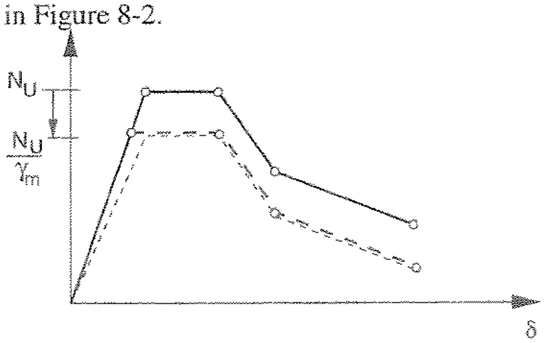


Figure 8-2 Inclusion of material factors on resistance characteristics

8.3.3 Method B

In this method the characteristic resistances are used in the analyses but without multiplying with the resistance factor or dividing with the material factor. The procedure to be followed will be:

1) Use characteristic strength (e.g. f_y) as input.

Divide the action factors γ_f with the resistance factors φ or multiply the action factors γ_f with the material factor γ_m to give combined safety factors (CSF) e.g. $\gamma_c=\gamma_m \cdot \gamma_f$ or $\gamma_c=\gamma_f/\varphi$.

Apply actions factored with the combined action factor for the groups of non-environmental actions (e.g. permanent loads).

Increment the characteristic environmental actions by applying an Environmental Load Multiplier (ELM) until the maximum structural capacity is reached.

Compare the value of the ELM with the required combined safety factor (CSF) for the environmental actions. In general both load combinations (ordinary and extreme in API or a and b in NPD) need to be checked.

For a safety format according to API LRFD /3/ the inequality (8.1) may then be written as:

$$R(R_{Mem}, R_{Jnt}\frac{\varphi_{Jnt}}{\varphi_{Mem}}, R_{Soil}\frac{\varphi_{Soil}}{\varphi_{Mem}}) \geq \frac{\gamma_P}{\varphi_{Mem}}P + \frac{\gamma_L}{\varphi_{Mem}}L + \frac{\gamma_E}{\varphi_{Mem}}E \qquad (8.4)$$

In the case of a safety format as given by NPD this method may be illustrated by the following equations.

$$R(R_{Mem}, \frac{R_{Jnt}}{\gamma_{Jnt}/\gamma_{Mem}}, \frac{R_{Soil}}{\gamma_{Soil}/\gamma_{Mem}}) \geq \gamma_{Mem}\gamma_D P + \gamma_{Mem}\gamma_L L + \gamma_{Mem}\gamma_E E \qquad (8.5)$$

As an example the use of NPD partial safety coefficients

γ_D = 1.0
γ_L = 1.0
γ_E = 1.3

γ_{Mem} = 1.15
γ_{Int} = 1.15
γ_{Soil} = 1.3

the inequality will reduce to:

$$R(R_{Mem}, R_{Jnt}, \frac{R_{Soil}}{1.13}) \geq 1.15 \cdot (P + L) + 1.5 \cdot E \qquad (8.6)$$

Structures meeting these requirements have safety levels comparable to the requirements set down for new designs.

In the non-linear analysis, the soil capacity is divided by 1.13 (ratio between material factor for soil and material factor for structure). All non-environmental loads are incremented to a load factor of 1.15. Environmental loads for the loading direction in question will then be incremented by the ELM until collapse of the platform.

The criterion is that the ELM value should be less than the CSF of 1.5 for the structure and 1.5·1.13 for the soil.

Again, care should be taken that the material factors only affect the *strength* of the component, and does not affect the *stiffness*.

8.4 API WSD format

Working stress design (WSD) or allowable stress design format is the design format used by the dominant marine codes since the start of the offshore oil activities. The working stress edition of API RP 2A /1/ is still the most used code for offshore structures.

With the working stress method the stresses are calculated according to a prescribed procedure. This procedure follows mainly the theory of linear elasticity. It is therefore impossible to use non-linear methods for checking of structures according to the standard method in this code.

However API RP2A is extended with a separate part (section 17), which covers reassessment of structures. In this chapter non-linear ultimate strength analyses are specially addressed.

The strength requirement is here formulated by use of the reserve strength ratio (RSR) concept. The RSR is defined in this API document as follows:

The reserve strength ratio (RSR) is the ratio of a platforms ultimate lateral load carrying capacity to its 100-year environmental condition lateral loading, computed using present API Recommended Practice 2A procedures.

In this definition, the characteristic resistance is taken as " best estimate" and not a lower bound or e.g. 5% fractile value.

The procedure will be as follows:

1) Use best estimate formulations for resistance as input.
2) Increment non-environmental loads to their specified value.
3) Increment the 100 year environmental loads applying an Environmental Load Multiplier (ELM) until the maximum structural capacity is reached.
4) Compare the value of the ELM with the required RSR.

The use of the best estimate or mean values for the resistance in comparison to lower bound or 5% fractile characteristic values will for the same RSR value imply a higher probability of failure. This should be taken into account when RSR values are compared with other safety factors. Furthermore the RSR safety format is developed for structures or structural components with a definite portion of environmental loads and will give low structural reliability values for structures or structural components dominated by permanent loads. This is discussed in the Commentary.

9 Software Requirements

9.1 General

The fundamental requirement of the software is that it should adequately represent the relevant failure modes for the basic components in framed offshore structures:

- Members
- Joints
- Foundation
- Loading

The software should have clear documentation as to which facilities are available and how they should be applied.

The software should include specifications of any limits of validity for special features, e.g. D/t-limits for a particular local buckling formulation, β-range for a joint capacity formulation etc.

9.2 QA requirements

It is essential that the software can document compliance with theoretical solutions and test results for single components, sub-structures and structural systems.

If there is any doubt about the formulation, a simple model should be subjected to a well-defined load and deformation path. This will allow the results to be judged and calibrated against engineering practice.

9.3 Input requirements

Simple program input reduces the possibilities for modelling errors and errors in result interpretation.

The input should be given in familiar engineering terms. Unfamiliar or specialised input parameters increase the possibilities for input errors.

The software should include pre-processing tools and default parameters to reduce the need for detail information from the user. This is especially relevant for specialised information outside the main engineering focus, e.g. parameters concerning numerical integration, mathematical stability or detail parameters for special program features.

Program default parameters should be listed with a description of what they imply and what any variation may represent.

9.4 Results presentation

The primary and essential validation of non-linear analysis results comes from understanding the development of the global collapse mechanism.

The software should present the analysis results in an efficient manner, such that the structural behaviour is easily understood by the engineer and is readily conveyed to others. Extensive use of computer graphic capabilities is recommended.

Identification of critical members should be made along with documentation of their strength (e.g. buckling load).

The software should contain self-checking mechanisms, such that clear indications are given if the analysis results at any stage in the analysis violate basic assumptions of the theory.

9.5 Minimum technical requirements

9.5.1 General

The following is a list of general modelling requirements for non-linear analysis of framed offshore structures. The treatment of different failure modes will vary from formulation to formulation. Different failure modes may typically be treated at one of the following levels:

1. As specialised features implemented in the program. E.g. local buckling criteria implemented in the program, including dent growth and modification of post-buckling load shedding.
2. As modelling guidelines. E.g. describing how the program's input parameters should be modified to capture the appropriate reduction in axial capacity and the accelerated load shedding in the post-collapse range.
3. As a provision for separate, manual checking after the analysis in completed.

Program modules separate from the structural analysis module are often used to calculate soil parameters and environmental loading. The interface between the modules should then be well defined and clearly documented, to prevent user errors or misunderstandings during transfer of data.

9.5.2 Material properties

- Yielding / yield hinges
- Strain hardening
- Strain rate effects

9.5.3 Section properties

- First fibre yield
- Gradual plastification of cross-section
- Fully plastic capacity
- Interaction between axial force and bending capacity
- Strain hardening

9.5.4 Member properties

General

- Elastic
- Compression (crushing) failure
- Yield (tension) failure
- Stability failure
- Post-collapse behaviour

Behaviour modes

- Beam bending
- Column Buckling
- Residual stresses / initial imperfections
- Member ductility
- Local buckling
- Hydrostatic Pressure

Special formulations

- Dented members
- Cracked members
- Grouted members
- Cyclic degradation

9.5.5 Tubular joint properties

Formulae

- API
- HSE
- User defined
- mean
- characteristic

Behaviour modes

- Elastic flexibility
- Ultimate Capacity
- Non-linear deformation

Special formulations

- Grouted joints
- Ring-stiffened joints
- Cracked joints
- Ground joints
- Cyclic degradation

9.5.6 Foundation properties

Behaviour modes

- Lateral soil failure
- Axial failure
- Monotonic behaviour
- Fully degraded behaviour

9.5.7 General FE modelling

General

- Joint eccentricities
- Internal hinges
- Linear dependencies
- Shim elements
- Locked-in-forces
- Linear springs
- Non-linear springs
- Pinned supports
- Fixed supports
- Spring supports
- Prescribed displacement
- Prescribed acceleration

9.5.8 Load modelling

General

- Load combinations
- Concentrated nodal loads
- Linearly distributed member loads
- Thermal loading
- Environmental Loading
- Self-weight calculated from density and section properties

Wave kinematics

- Stokes 5th
- Airy
- Wheeler
- Stream function
- Current loading
- Buoyancy Loads
- Marine Growth

Loading algorithms

- Initial loads (self-weight and buoyancy)
- Proportional loads
- Non-proportional loading
- Wave height incrementation
- Wave-in-deck forces
- Cyclic storm loading

10 Reporting

10.1 General

This chapter contains listing of the main elements to be observed and presented. Particular advice based on experience is also included.

The aim is to keep the report as compact as possible, including the most vital information and leave detailed information to appendices.

A typical layout could look like this:

- An executive summary with main results
- An introduction, giving a short description of the problem
- Basis for the analysis; i.e.: presentation of the analysis method, acceptance criteria, failure-criteria, etc.
- Modelling
- Analysis and Results
- Attachments, modelplots, resultlistings, model and load verification etc.

In the following the elements and extent of reporting is presented.

10.2 Introduction and summary

The Introduction shoud include the following information:

- Description of problem;
  - location
  - platform
  - analysis goals
  - areas of concern
- Analysis method to be used

The Executive Summary should include:

- Main result, (environmental load multiplier, collapse mechanism, etc.)
- areas of concern
- conclusive statement

10.3 Basis for the analysis

10.3.1 General

This chapter is a collection and systemisation of the data necessary to perform the analysis.

10.3.2 Analysis method

Description of the actual non-linear method

An extract from the analysis method description can be given here. The major features should be listed:

- load application
- solution control
- analysis strategy

10.3.3 Failure criteria

Depending on software and type of structure, a description on how relevant failure criteria are included by the programme should be given.

The issues described in 4.2 and 4.3 should be considered and documented.

10.4 Modelling

10.4.1 Structural modelling

Frame Modelling

The frame geometry should be modelled in accordance to description in section 5.2.

The geometrical model should be presented with an overall plot. A complete description of the model, including plots showing members and joints should be presented in appendices, or referred to elsewhere. Any modification of the model, such as additional risers, damaged members, reinforcement of structure, etc., should be documented.

The following issues have been described in this guide, and should be considered and documented if appropriate:

- pile connectivity
- grouted Piles
- conductors/risers
- conductor connectivity
- joint offsets
- joint flexibility
- grouted joints
- cracked joints
- ground joints
- dented members
- member imperfections
- grouted members
- yield strength
- strain hardening
- strain rate effects
- locked-in-forces
- corrosion allowance

Foundation modelling

The non-linear soil data should be listed. More detailed information should be documented in appendices or other references.

The non-linear soil-model should be described with figures and data.

Member modelling

Depending on type of analytical technique, the member behaviour should be documented.

The following issues should be considered, and documented according to description in section 5.2.1 to 5.2.8.

The documentation should be kept short, and it might be appropriate to refer to appendices.

- FE-selection
- Material modelling
- Cross-sectional modelling
- Column buckling
- Local Buckling
- Dented members
- Member ductility
- Cyclic degradation

Joint modelling

The screening procedure performed to include representation of failure should be described. Joints with limited capacities should be listed, and/or shown on figures. The limiting capacities of the considered joints should be presented in appendices.

In addition the following issues should be considered, and documented according to description in section 5.4.2 to 5.4.9.

- Joint flexibility
- Joint eccentricities
- Description and verification of joint failure approach.
- Failure criteria; capacity and flexibility
- Cyclic Degradation (failure caused by through thickness cracking)

10.4.2 Load modelling

Permanent Loads, and Live Loads

A summary of the permanent and live loads should be presented in a table. Load modifications should be documented.

Environmental Loads

Load sum, and overturning moment for each directions should be documented. Design basis for wave load generation should be given either here, or in appendices.

Load Combinations

Load combinations should be described.

Application of Loads

The procedure for determining the should be presented.

The environmental load multiplier can be determined either on the basis of increasing the characteristic environmental loading, or by increasing the wave height.

10.5 Analysis and result

Analysis

The loads are to be applied in a logical sequence, and incremented until the failure mechanism is identified. It must therefore be documented that the numerical stability is acceptable throughout the analysis.

Results

For ultimate strength analysis, the structural performance is described by the global collapse mechanism, the system capacity and the sequence of non-linear events.

In this chapter only the main results should be presented, containing the following information for each loadcombination:

- description of the collapse mechanism
- system capacity
- sequence of events (yield, plasticity, buckling, etc.)

A complete list of all components that are failing according to criteria as referenced in 9.3.3 during the analysis should be presented. In addition the following documentation should be included:

- global P-D plot
- P-D plot of important members experiencing yield/buckling
- deformed structure

More detailed result listings should be presented in appendices.

10.6 References

The reference list should as a minimum contain the following:

1. ULTIGUIDE (this document)
2. Theory/User Manual of actual analysis method.
3. Design premises or specifications

10.7 Appendices

All appendices referred to in the report.

- Model plots and listing
- Model Verifications (structure, member, joint, foundation)
- Load verifications
- Listing of results

11 References

/1/ ISO 13819-1 Petroleum and natural gas industries- Offshore structures Part 1: General requirements. First edition 1995.

/2/ API RP 2A WSD Recommended Practice for Planning , Designing and Constructing Fixed Offshore Plattforms Working Stress Design. 20th edition, 1993 with supplements 1996.

/3/ API RP 2A LRFD Recommended Practice for Planning , Designing and Constructing Fixed Offshore Plattforms —Load and Resistance Factor Design, First Edition 1993

/4/ ISO 13819-1 Petroleum and natural gas industries- Offshore structures Part 2: Fixed Steel Structures, Draft C, July 1997

/5/ NORSOK N-004 Design Of Steel Structures, Annex A Design Against Accidental Actions, December 1998.

/6/ Inge Lotsberg, DNV Report No. 97-3263. JIP Ultiguide Phase 2. Strain Criteria for Ultimate Strength Analysis

/7/ Bjørn Skallerud, SINTEF Report No. STF 22-F97723 Ultiguide Phase 2. Low Cycle Fatigue Criteria for Capacity Assessments of Offshore Steel Platforms.

/8/ Gudmund Eiksund, SINTEF Report No. STF 22-F97667 Pile Soil Modelling for Non-linear Pushover Analysis.

/9/ NPD Regulations relating to loadbearing structures etc. Issued by the Norwegian Petroleum Directorate 7 February 1992

/10/ Hellan, Ø.: "Non-linear Pushover and Cyclic Analyses in Ultimate Limit State Design and Reassessment of Tubular Steel Offshore Structures" MTA report (Ph.D.) 1995:108, Norwegian Institute of Technology, Trondheim, Norway, 1995

/11/ Stewart, G. and Tromans, P.S.: "Nonlinear Reassessment of Jacket Structures under Extreme Storm Cyclic Loading.
Part II – Representative Environmental Load Histories" *Proc. Offshore Mechanics and Arctic Engineering conference (OMAE), Glasgow, Scottland, 1993.*

/12/ Hellan, Ø., Tandberg, T. and Hellevig, N.C.: "Nonlinear Reassessment of Jacket Structures under Extreme Storm Cyclic Loading.
Part IV – Case Studies on Existing Nort Sea platforms" *Proc. Offshore Mechanics and Arctic Engineering conference (OMAE), Glasgow, Scotland, 1993.*

/13/ Skallerud, B., Amdahl, J., Johansen, A. and Eide, O.I.: "Cyclic capacity of tubular beam-columns with local buckling: numerical and experimental studies" *Proc. Offshore Mechanics and Arctic Engineering conference (OMAE), Florence, Italy, 1996.*

/14/ Øyvind Hellan, Arnfinn Reitan, SINTEF Report No. STF22-F98685, DNV Report No. 98-3097. JIP Ultiguide Phase 2. Evaluation report on dynamic effects

/15/ PD6493:1991,"Guidance on Methods for Assessing the Acceptability of Flaws in Fusion Welded Structures", British Standard Institution, Second Edition, August 1991

/16/ Øyvind Hellan, SINTEF Report No. STF22-F97712, Representation of fabrication tolerances and residual stresses in nonlinear pushover analyses

/17/ Øistein Hagen, Gudfinnur Sigurdsson DNV Report No. 98-3136 Structural Reliability Aspects

/18/ Zimmerman, T. J. E., Stephens, M. J., DeGeer, D. D. and Chen, Q.:"Compressive Strain Limits for Buried Pipelines", 1995 OMAE - Volume V, Pipeline Technology ASME.

/19/ Fessler, H., Mockford, P. B., Webster, J. J. Parametric Equations for the Flexibility Matrices of Single Brace Tubular Joints in Offshore Structures. Proceedings of the Institution of Civil Engineers, p 659-673, London 1986

/20/ Fessler,h.,Mockford P:B:, Webster, J. J., Parametric equation for the Flexibility Matrices of Multi-Brace Tubular Joints in Offshore structures. Proceedings of the Institution of Civil Engineers, p 675-696, London 1986

/21/ Buitrago, J., Healy, B. E., Chang, T. Y. Local Joint Flexibility of Tubular Joints, Proceedings of the 12th International Conference on Offshore Mechanics and Arctic Engineering, OMAE 1993

/22/ Chen, B. Hu, Y. Tan, M. Local Joint Flexibility of Tubular Joints of Offshore Structures, Proceedings of the Ninth International conference on Offshore Mechanics and Arctic Engineering Volume III 1990

/23/ Johnston, B.G. (Ed.) "Guide to Stability Design Criteria for Metal Structures", Third Edition, Structural Stability Research Council, John Wiley & Sons, New York (1976)

/24/ AISC "Manual of steel construction, load and resistance factor design". American Institute of steel construction, 1st ed. (1986).

/25/ Massonet, C. (1959) "Stability considerations in the design of steel columns", Journal of the Structural Division, ASCE, vol. 85

/26/ Austin, W.J (1961) "Strength and design of metal beam-columns", Journal of the Structural Division, ASCE, vol. 87

/27/ Roik, K. (1978). "Vorlesungen über Stahlbau." Verlag von Wilhem Ernst & Sohn, Berlin, Germany

/28/ Galambos, T.V. (Ed.) "Guide to Stability Design Criteria for Metal Structures" , 4th Edition, Structural Stability Research Council, John Wiley & Sons, New York (1988)

/29/ Holmås, T, Remseth, S. and Hals, T.E."Approximate Flexibility and Stress Analysis of Tubular Joints in Marine Structures", Report STF71 F85045, SINTEF Structures and Concrete, Trondheim, Norway (1985)

/30/ Holmås, T. (1987) "Implementation of Tubular Joint Flexibility in Global Frame Analysis", Division of Structural Mechanics Report No. 87-1, The Norwegian Institute of Technology, Trondheim

/31/ Efthymiou, M (1985). "Local rotational stiffness of unstiffened tubular joints". Report RKER.85.199, Shell Research, Rijswijk, The Netherlands

12 Commentary

Comm. 3.1 Non-linear methods in ultimate strength analysis

Current structural analysis practice for fixed offshore structures is based on linear-elastic analyses of the structure, combined with ultimate strength criteria applied at component level.

Instead of separating the frame analysis and the component strength / stability checking, the non-linear analyses treat the system as a whole, including the separate failure modes, the interaction between the frame behaviour and the individual components, and the impact of one component failure on the remaining system. The analyses provide detailed information about the collapse mechanism of a structure, usually following an event-to-event strategy, tracing first fibre yield, occurrence of plastic hinges and failure of each member, until and beyond the maximum capacity of the structure is reached.

When the load carrying capacity of one component (member / joint) is reached, the loads will seek alternative paths and, if alternative load paths exists, lead to successive component "failures" until a complete mechanism is formed and the structure collapses. If the loads cannot find alternative load paths, then structural collapse will coincide with first member (component) failure.

The physical failure modes are the same as in conventional procedures, and the engineering knowledge required to evaluate the structural performance is the same. The main difference lies in the actual code checking format. Instead of performing code checking as a separate activity after the frame analysis, the component failure criteria form an integral part of ultimate strength analysis.

Once the component failure modes are sufficiently verified, the FEM solution itself should provide consistency between results on component level and systems level.

Comm. 3.3 Software validation

Software validation is of course also the responsibility of the program vendor, but the ultimate responsibility for the structure and suitability of the modelling will still lie with the engineer.

Comm. 4.1 General

Non-linear pushover analyses may be used to predict states prior to structural collapse which for simplicity are classified as ultimate limit states given. Often referred as first component failure. Upon violation of one component elastic limits, the loads will seek alternative load paths and, if alternative load paths exists, lead to successive component "failures" until a complete mechanism is formed and the structure collapses. If the loads cannot find alternative load paths, then structural collapse will coincide with first member (component) failure.

First component failure as definition of an ultimate limit state is strongly related to that of current design methods. The latter limit state is caused by failure of one or more components in sequence.

To apply non-linear pushover analyses in design and integrity assessment of offshore structures, it is important that the analysis tool can document compliance with established design formulations.

Once the component failure modes are sufficiently verified, the FEM solution itself should provide consistency between results on component level and systems level (redistribution of forces from a failed components to the adjacent structure).

Comm. 4.2 Members

The member behaviour predicted by non-linear analysis programs should represent observed behaviour from tests and theoretical solutions.

Comm. 4.2.3 Column buckling

The buckling problem has traditionally been approached from different angles, leading to different design equations and column curves. The rationale behind the column curves differ, and although most column curves have been verified against extensive test data and practical experience, no buckling curve can be said to represent the 'true' behaviour of a compression member. Present column curves can rather be regarded as pragmatic formulae for the assessment of the member capacity, based on test data or numerical simulations for different cross sections. Most of today's column curves reflect given confidence levels (e.g. lower 5% percentile) of the background database, and some sources present separate column curves for mean column strength and characteristic (lower bound) column strength.

Thus, choice of column curve also implies choice of the 'safety level' inherent in the curve.

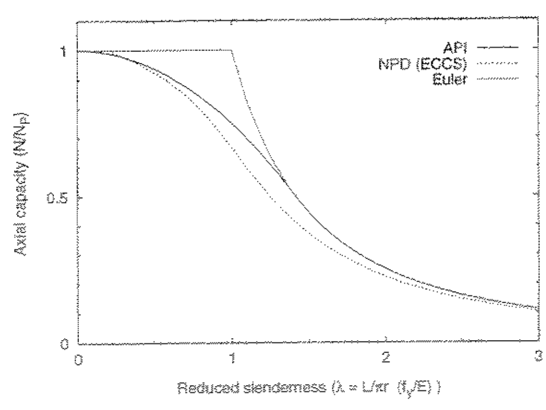


Figure 12-1 Column buckling curves

Critical load theory

The strength of a perfectly straight prismatic column was originally established by Euler in 1759 as

$$P_E = \frac{\pi^2 EI}{L^2} \tag{12.1}$$

provided the material is still elastic when buckling occurs. At this load, buckling takes place through bifurcation from the ideally straight shape, and the column starts to deform laterally. The Euler theory was extended to inelastic buckling by Engesser in 1889 and Shanley in 1946, introducing the tangent modulus and reduced modulus concepts. Early column curves were based on either the tangent modulus theory, or test results for allowable stresses in columns. The Steel Structures Research Council (SSRC) (formerly Column Research Council, CRC) in 1960 published their column curve of the form

$$P_E = N_P - B\left(\frac{KL}{i}\right)^2 \tag{12.2}$$

where K is the effective length factor of the column, i is the radius of gyration and Np is the plastic axial capacity of the column section. This parabola was chosen because it represented an approximate median between the tangent modulus strength of a wide flange column about the strong and the weak axes /23/. The column strength in the elastic range is represented by the Euler formula. B is a curve-fitting constant. The point of demarcation between elastic and inelastic behaviour was chosen to be $f_p = 0.5 f_y$, because this was a conservative measure of the proportionality limit for hot-rolled wide flange I shapes.

These deliberations led to the following column curve, which is still in wide use by the industry

$$\frac{P_{Cr}}{N_P} = \begin{cases} 1 - \dfrac{\bar{\lambda}_k^2}{4} & , \quad \bar{\lambda}_k \le \sqrt{2} \\ \dfrac{1}{\bar{\lambda}_k^2} & , \quad \bar{\lambda}_k > \sqrt{2} \end{cases} \tag{12.3}$$

where

$$\bar{\lambda}_k = \sqrt{\frac{N_P}{P_{Ek}}} = \frac{iKL}{\pi\sqrt{E/f_y}} \tag{12.4}$$

Imperfect column concept

Most modern column curves are based on the imperfect column concept, representing the 'real', observed capacity of columns with existing geometric imperfections and residual stresses. Thus, column curves differ from the Euler buckling curve, except for very large slenderness. Several different formats of the curve are in use. The AISC-LRFD Specification of 1986 (Appendix A.2) /24/ recommends a single column curve for all section types. However, results from tests on steel columns with different cross sectional shapes have shown that the strength varies considerably. These observations have lead to the concept of multiple column curves. Such curves have been developed through research performed at Lehigh and test performed in Europe under the direction of ECCS.

$$\frac{P_{Cr}}{N_p} = \begin{cases} 1 & ,\bar{\lambda}_k \le 0.2 \\ \dfrac{\varphi - \sqrt{\varphi^2 + \bar{\lambda}_k^2}}{2\bar{\lambda}_k^2} & ,\bar{\lambda}_k > 0.2 \end{cases} \qquad (12.5)$$

where

$$\varphi = 1 + \alpha(\bar{\lambda}_k - 0.2) + \bar{\lambda}_k^2 \qquad (12.6)$$

Column curves for different shapes are distinguished by different α-parameters, with α= 0.21 for tubular members. α-parameters for other section types are listed in Table 12-1.

Table 12-1 Multiple column curves

Curve	α	Description
A0	0.13	High-strength steel sections
A	0.21	Stress-relieved shapes; tubular and RHS sections; I-sections of high-grade steel
B	0.34	Hot-rolled light and medium sections; welded H-sections bent about the major axis
C	0.49	Heavy rolled shapes; T, Channel and compact shapes
D	0.76	Rolled heavy W and welded heavy H bent about weak axis

ISO column curve

Recent developments by ISO /4/ recommend the following column curve:

$$\frac{P_{Cr}}{N_p} = \begin{cases} 1 - 0.28\bar{\lambda}_k^2 & , \quad \bar{\lambda}_k \le 1.34 \\ 0.9/\bar{\lambda}_k^2 & , \quad \bar{\lambda}_k > 1.34 \end{cases} \qquad (12.7)$$

Beam-column stability

The term beam-column denotes a member subjected to a combination of axial force and bending moments. One approach to beam-column design formulae is the simple interaction formula

$$\frac{N}{P_{Cr}} + \frac{M}{M_U} \le 1 \qquad (12.8)$$

where P_{Cr} is the critical column buckling load as described by a column curve, and M_U is the ultimate bending moment in absence of axial loads. End bending moments, shear forces and lateral concentrated and/or distributed forces will produce a primary bending moment M_0 and a primary deflection δ_0. The axial force will act on the primary deflection and produce additional, second-order bending moments and deflections as illustrated in Figure 12-2. The second-order moment and deflection are often called P-δ effects or member instability effects.

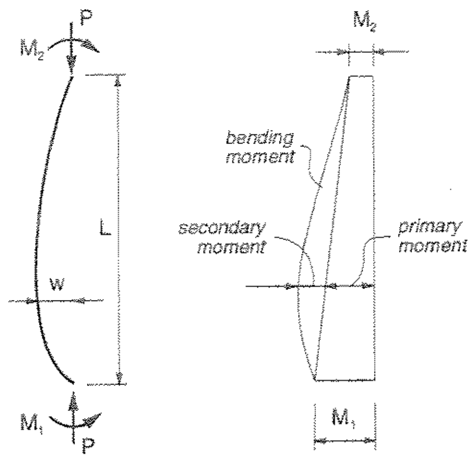


Figure 12-2 Second-order moment and deflection

By assuming the secondary moment (and deflection) as a half sine wave with the maximum deflection at midspan, the total deflection can be expressed by

$$\delta = \frac{1}{1 - N/P_E}\delta_0 \tag{12.9}$$

Assuming the maximum primary moment occurs at midspan, the total maximum moment is given as

$$\begin{aligned} M_{Max} &= M_{0,\max} + N \cdot \delta_0 \frac{1}{1 - N/P_E} \\ &\approx \frac{C_m}{1 - N/P_E} M_{0,\max} \end{aligned} \tag{12.10}$$

C_m is called the 'equivalent moment factor', and $C_m / (1 - N/P_{Ek})$ is called the second-order moment amplification factor or the P-δ amplification factor.

Inserting (12.10) into (12.9) gives the following format for the interaction formula

$$\frac{N}{P_{Cr}} + \frac{C_m M}{(1 - N/P_E)\, M_U} \leq 1 \tag{12.11}$$

In conventional design procedures, the maximum field moment is checked by (12.11), maximum end moments are checked separately by (12.9).

The equivalent moment concept

Equation (12.11)is an approximate measure to introduce second-order effects in the design. Instead of performing a second-order analysis, bending moments from linear analyses are multiplied by C_m $/ (1 - N / P_{Ek})$ to estimate the 'real' first order + second order moment in the beam-column.

The concept of equivalent moment replaces the design of a beam-column subject to an arbitrary combination of end moments with the design of an equivalent beam column subject to equal and opposite end moments of magnitude $M_{eq} = C_m M_B$ (Figure 12-3).

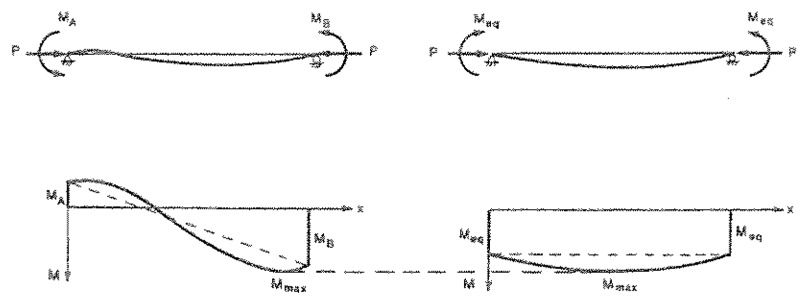


Figure 12-3 Equivalent moment concept

The C_m factor is easily determined for simply supported beam-columns subjected to end forces (Equation (12.12)). For other end restraints and loading conditions calculation of second-order magnification effects is more elaborate, and the total maximum moment is not readily obtained.

$$C_m = \frac{\sqrt{(M_A / M_B)^2 + 2(M_A / M_B)\cos\left(\pi\sqrt{P / P_E}\right) + 1}}{\sqrt{2(1 \ \cos\left(\pi\sqrt{P / P_E}\right))}} \qquad (12.12)$$

Approximate expressions for use in design have been presented e.g. /25/and /26/.

$$C_m = \sqrt{0.3(M_A / M_B)^2 \ 0.4(M_A / M_B) + 0.3} \qquad (12.13)$$

$$C_m = 0.6 - 0.4(M_A / M_B) \geq 0.4 \qquad (12.14)$$

Recent developments by ISO recommend the following C_m factor

$$C_m = 1.0 - 0.4\frac{N}{P_E} \qquad (12.15)$$

The effective length concept

The isolated column is a theoretical concept; it rarely exists in practice. Usually, a column forms part of a structural frame and its stability is interrelated with the stability of the entire structure. The structure imposes not only axial forces but also end restraints and flexural or torsional forces on the column.

The effective length concept separates the column stability from frame stability and reduces the design problem to that of an isolated member with given forces and end restraints. The "effective length" *KL* of a column defines part of the buckled deflection between points of zero curvature. I.e. the "effective length" is the length of a fictitious hinged-end column that would have the same Euler bifurcation load as the actual restrained column.

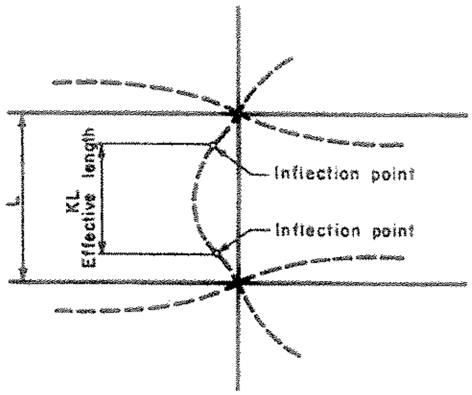


Figure 12-4 Effective length concept

Effective length factors for any framed member can be determined by eigenvalue analysis, if the restraint conditions are known. Alternatively, the effective length factor can be read from curve charts /27/ or alignment charts as shown in Figure 12-5.

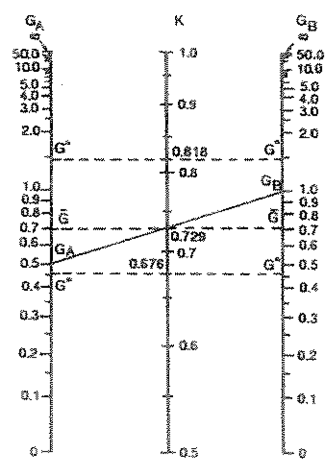


Figure 12-5 Alignment chart for braced frames

Alignment charts like the one shown in Figure 12-5 were originally developed for idealised building structures. They are based on a number of assumptions that have limited validity for existing offshore structures. Among these are the assumptions of linear elastic column behaviour, completely rigid joints and no axial loads in the members that supply the rotational end restraints. All columns at one level are assumed to buckle simultaneously, inducing the failure pattern shown in Figure 12-6, corresponding to a rotational restraint of $2EI/L$ of all restraining members /23/.

These assumptions are generally not fulfilled, and several adjustment factors have been presented to extend the alignment chart to non-idealised cases /28/.

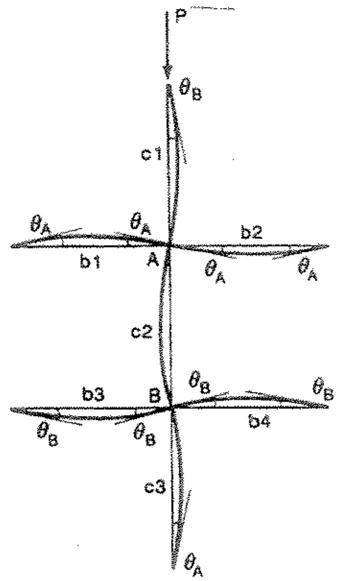


Figure 12-6 Assumed failure pattern for alignment chart

Since compression members fail through elasto-plastic buckling rather than Euler buckling, and since most beam-columns also carry some degree of transverse loading, use of the effective length concept is clearly an approximate procedure. Most codes accept the use of effective length factors determined through refined analysis of the beam-column in question. Such an analysis should consider the joint restraints, joint flexibility and joint sidesway. Furthermore, the joint must have sufficient capacity to actually impose the calculated end restraints at the required load level. In lieu of such refined analyses, most codes present recommended values for typical bracing configurations. These values are conservative, often based on an underlying assumption of a fully optimised truss work where buckling stresses in compression members and yield stresses in tension members are reached at the same level of live load /28/. On this basis, no restraint would be supplied at the joint, and the effective length factor is selected as for a member with pinned end(s).

Comm. 4.2.4 Tubular section capacities

Local buckling

For tubular structures, limits to the sectional slenderness are defined in terms of D/T or the non-dimensional section slenderness parameter $\alpha = D/T \cdot f_y/E$. To account for interaction between local buckling and column buckling, the section yield load is replaced by the inelastic buckling load in the column curves and beam-column capacity equations. Equation (12.16) shows the compactness criterion for local buckling of tubular members under axial loading specified by API /3/.
Table 12-2 ECCS cross section classification lists the section slenderness requirements of ECCS for tubular members.

$$\frac{N_{Ux}}{N_P} = \begin{cases} 1 & , D/T \leq 60 \\ 1.64\ 0.23\sqrt[4]{D/T} & , D/T > 60 \end{cases} \qquad (12.16)$$

Table 12-2 ECCS cross section classification

Cross section class	Classification	$\alpha = D/T \cdot f_y/E$
1	Plastic cross sections	$\alpha_C < 0.056$
2	Compact cross sections	$\alpha_C < 0.0783$
3	Semi-compact cross sections	$\alpha_C < 0.1007$
4	Slender cross sections	$\alpha_C > 0.1007$

Most offshore codes allow for utilisation of the full plastic bending moment for compact sections. Recent developments by ISO recommend the following local buckling stress under axial loading (only)

$$\frac{N_{Ux}}{N_P} = \begin{cases} 1 & \bar{\lambda}_x \leq 0.17 \\ 1.047 - 0.274\bar{\lambda}_X & 0.17 < \bar{\lambda}_x \leq 1.91 \\ 1 \,/\, \bar{\lambda}_x & 1.91 < \bar{\lambda}_x \end{cases} \qquad (12.17)$$

where

$$\bar{\lambda}_x = \frac{N_P}{N_{Xe}} = \frac{0.6}{\alpha_C} = 0.6\frac{T}{D}\frac{E}{f_y} \qquad (12.18)$$

Bending

Equations (12.19) and (12.20) show the compactness criterion for tubular members under bending specified by API /3/ and NPD /9/, respectively.

$$\frac{M_U}{M_P} = \begin{cases} 1 & \alpha_c \leq 0.0492 \\ 1.13 - 2.58\alpha_C & 0.0492 < \alpha_c \leq 0.0985 \\ 0.94 - 0.76\alpha_C & 0.0985 < \alpha_c \quad 300 f_y/E \end{cases} \qquad (12.19)$$

$$\frac{M_U}{1.1 \cdot M_Y} = \begin{cases} 1 & \alpha_c \leq 0.0875_c \\ 1.07 - 0.8\alpha_C & 0.0875 < \alpha_C \end{cases} \qquad (12.20)$$

M_p is the full plastic bending moment, and M_Y is the bending moment at first fibre yield.

The capacity formula recommended by ISO is

$$\frac{M_U}{M_p} = \begin{cases} 1 & \alpha_c \leq 0.0517 \\ 1.13 - 2.58\alpha_C & 0.0517 < \alpha_c \leq 0.1034 \\ 0.94 - 0.76\alpha_C & 0.1034 < \alpha_c \quad 120 f_y/E \end{cases} \tag{12.21}$$

Combined axial compression and bending

Equations (12.22) show the strength criterion specified by API /3/ and ISO /4/. Equation (12.23) shows the strength criterion of NPD /9/.

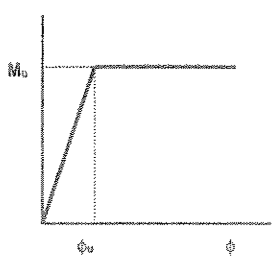
$$1 - \cos\left(\frac{\pi}{2}\frac{N}{N_{Ux}}\right) + \frac{M}{M_U} \leq 1.0 \tag{12.22}$$

$$\frac{N}{N_{Ux}} + \frac{M}{M_U} \leq 1 \tag{12.23}$$

Comm. 4.3.1 General

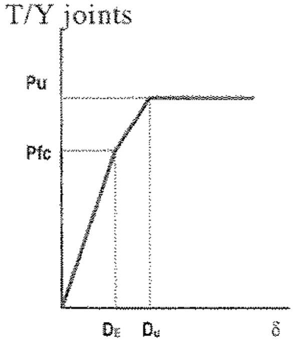
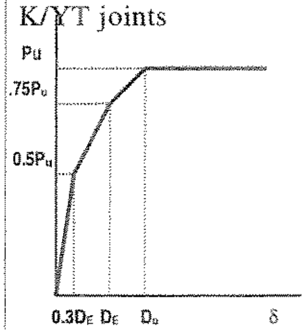
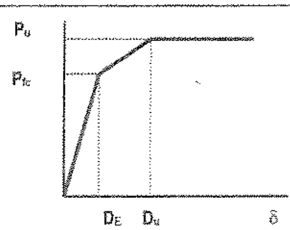
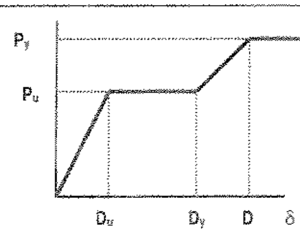
Recommendations for the representation of large-deflection joint flexibility as it affects the ultimate response of jacket framed structures are detailed below. The formulae have been derived from an evaluation of the tubular joint experimental database used in the derivation of the draft ISO standard for fixed steel structures. The mean capacity values (P_u) are given in the draft ISO code.

Table 12-3 Joint moment flexibility representation in ultimate strength analysis *

All joints	In-plane bending	Out-of-plane bending
	Mu (see ISO) $\phi_u = \left(\frac{98.4\times10^{-3}}{\beta^{2.75}}\right)\left(\frac{M_u \sin\theta}{D^2 F_y T}\right)$	Mu (see ISO) $\phi_u = \left(\frac{1.82\times10^{-8}\left(\frac{D}{T}\right)^{3.65}}{\beta^{6.08}}\right)\left(\frac{M_u \sin\theta}{D^2 F_y T}\right)$

* P_u etc based on mean capacity formulae presented in the Commentary to the ISO draft code; Units N, m; See also Nomenclature for definition of terms

Table 12-4 Joint axial flexibility representation in ultimate strength analysis *

Configuration	Tension	Compression
T/Y joints 	P_u (see ISO) $$D_u = \left(\frac{6.0\times10^{-3}\left(\frac{D}{T}\right)^{1.70}}{\beta^{0.71}K_a\sqrt{Q_\beta}}\right)\left(\frac{P_u \sin\theta}{DF_y}\right)$$ $P_E = P_{fc}$ (ISO mean) $$D_E = \left(\frac{1.09\times10^{-3}\left(\frac{D}{T}\right)^{1.92}}{\beta^{0.80}K_a\sqrt{Q_\beta}}\right)\left(\frac{P_E \sin\theta}{DF_y}\right)$$	P_u (see ISO) D_u as tension $P_E = 0.85\ P_u$ D_E as tension
K/YT joints 	P_u (see ISO) $$D' = \left(\frac{0.0209\left(\frac{D}{T}\right)^{1.225}}{\beta^{0.51}K_aQ_g\sqrt{Q_\beta}}\right)\left(\frac{P_u \sin\theta}{DF_y}\right)$$ $D_u = 0.5D'$ $$D_E = \left(\frac{2.78\times10^{-4}\left(\frac{D}{T}\right)^{1.85}}{\beta^{0.773}K_aQ_g\sqrt{Q_\beta}}\right)\left(\frac{P_u \sin\theta}{DF_y}\right)$$	P_u (see ISO) $D_u = D'$ D_E as tension
X joints (total deformation across joint)	 P_u (see ISO) $$D_u = 0.048\gamma\left(\frac{P_u}{DF_y \sin\theta}\right)$$ P_{fc} (see ISO) $D_E = 0.29\ D_u$	 P_u (see ISO) $$D_u = (0.118\gamma - 1.234)\left(\frac{P_u}{DF_y \sin\theta}\right)$$ $Dy = 0.5D$ $P_y = \pi dtF_y$

* P_u etc based on mean capacity formulae presented in the Commentary to the ISO draft code; Units N, m; See also Nomenclature for definition of terms

It must be recognised that understanding of tubular joint behaviour comes largely from isolated tests in which the laboratory simplifications may depart from reality. For example:

- The support to the chord(s)/brace(s) may be more or less flexible than frame continuity potentially affecting failure mode, capacity and flexibility.
- Only one mode of loading is generally applied. Although limited tests have enabled a force-moment interaction 'failure' surface to be defined, there is little data giving insight to the degree of deformation (flexibility) under combined loads.
- The rapid load shedding apparent in isolated tests may be less relevant within structure where the large deformations are prevented by the geometric constraints of the frame.

These simplifications reflect the complexity of the physical behaviour and result in uncertain knowledge regarding the large deflection response of tubular joints. It is therefore recommended that effort be focussed on identifying any zones where large deformations may arise, then including a simple joint model and finally assessing the sensitivity of the system response to reasonable variations in the modelling assumptions. In certain circumstances it may be appropriate to develop a detailed representation of joint response characteristics but, in general, the additional effort may not be appropriate or justified.

It is important to post-process the degree of local deformation developed in comparison with member diameters etc. Although gross deformations may be sustained under monotonic loads, the strain associated with the degree of geometric distortion and material plasticity may result in high-stress low cycle fatigue under reversing load or incremental plastic collapse. The ductility of the joint may be considered to be limited for practical purposes. If the ductility demand is greater this may impose a limit on the useful system capacity.

Explicit strain limits cannot readily be accounted for in ultimate strength analyses and therefore ductility limits for different joint types are given in terms of local deformation. Indicative values which may act as a trigger for closer investigations are:

- Axial deformation > 0.05D or 0.2g for K joints or 0.02D for X joints with $\beta>0.9$, whichever is lesser
- Rotation > 0.05 radians.

In selecting appropriate spring formulations, it should be noted that the non-linear response at a joint develops permanent deformations; the joint does not recover with reducing load, instead there is a permanent set with unloading typically corresponding with the initial elastic stiffness.

When using springs consideration should also be given to the coupling of responses to prevent all force and moment contributions being exploited to their maxima. A non-linear interaction based on the ISO strength formulation may be employed. Alternatively it may be adequate in terms of the effect on global response, simply to limit the growth in moment at the point the axial capacity is reached. Similarly, for combined loading modes at complex intersections, interpolation of capacity and associated deformations may be adequate.

Comm. 4.3.9 Joints with cracks

F_{AR} is the reduction factor to allow for loss of load-bearing cross-sectional area due to presence of the flaw and is given by the following equation:

$$F_{AR} = \left(1 - \frac{Crack\ area}{weld\ length * T}\right) * \left(\frac{1}{Q_\beta}\right)^{m_q} \tag{12.24}$$

Q_β allows for the increased strength observed at β values above 0.6. (β = diameter of brace/diameter of chord). Q_β is known as the geometrical modifier, usually used in design codes to account for the increasing capacity of uncracked tubular joints at high β:

- $Q\beta = 1$ for $\beta \leq 0.6$
- $Q\beta = 0.3/\beta(1-0.833\beta)$ for $\beta > 0.6$

m_q is the power allocated to Q_β and depends on the approach used to estimate the capacity of the uncracked joint:

- for tubular joints containing part-thickness flaws, $m_q = 0$
- for tubular joints containing through-thickness flaws, validated correction factors giving lower bound estimates of the collapse load are at present limited to joints with β ratios less than 0.8 and the following configurations:
  - K-joints with a through thickness crack at the crown subjected to balanced axial loading
  - axially loaded T and DT joints with a through thickness crack at the saddle.

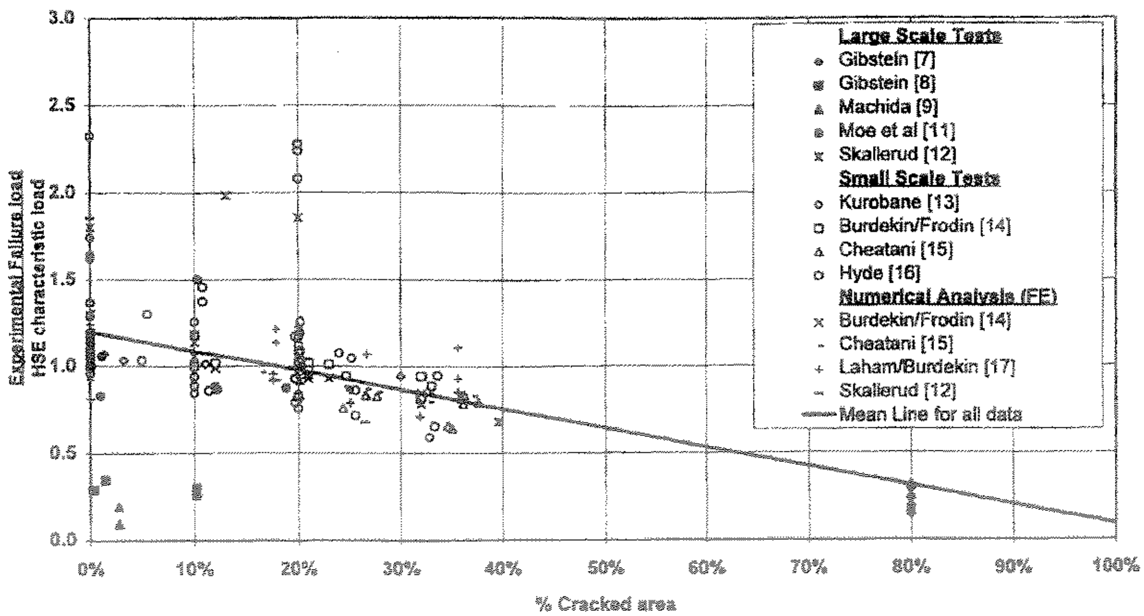


Figure 12-7

For K joints subjected to balanced axial loads, the revised BS PD 6493 procedure recommends the use of

- the Health and Safety Executive characteristic compression design strength with $m_q = 1$, or
- the API RP2A compression design strength (omitting the safety factor of 1.,7) with $m_q = 0$.

For D and DT joints, the revised BS PD 6493 procedure recommends the use of

- the Health and Safety Executive characteristic tension design strength with $m_q = 1$, or
- the API RP2A tension design strength with $m_q = 0$.

Comm. 4.3.10 Cyclic loading

Only limited information is available on the behaviour of tubular joints under severe cyclic loading. However, the available test data indicate that only very limited cyclic degradation (if any) will occur in a tubular joint under extreme storm cyclic loading, as long as the peak in the cyclic load history does not exceed the monotonic ultimate joint capacity.

In other words: present findings does not indicate any need for separate checks of cyclic joint behaviour unless the joint is loaded beyond its ultimate capacity, i.e. into the post-peak region.

Comm. 4.4.2 Cyclic behaviour

Figure 12-8 illustrates different behaviour modes of a (general) structure under cyclic loading. Cyclic loading may lead to ductility exhaustion, causing critical components to fracture. This may in turn trigger global instability and failure of the whole structure, as indicated by the left-hand curve in Figure 12-8. Alternatively, the component remains intact, but global deformations in each cycle increase until they are no longer tolerable. This is called incremental collapse, illustrated by the central plot. The final plot indicate a case where the deformations in each cycle decrease until, eventually, the structural behaviour is stabilised and further load cycles only lead to elastic response. This (desirable) state is called shakedown.

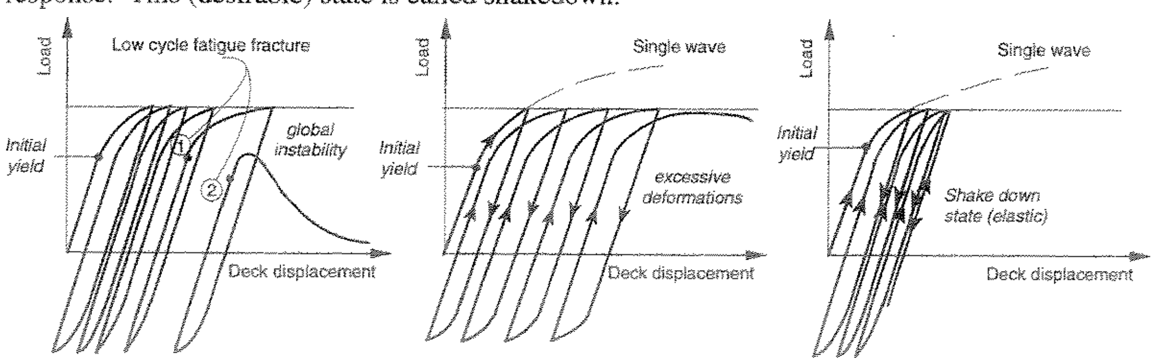


Figure 12-8 Failure and survival modes under cyclic load

Recent research projects /11/ /12/ indicate that cyclic degradation has little practical consequence for fixed offshore structures under normal utilisation ratios.

However, if a structure is utilised beyond first member buckling or beyond ultimate joint capacity (exploiting system strength effects), separate checks of cyclic degradation should be performed. It should be checked whether the highly utilised sections yield under both forward and reverse wave loading. If this is the case, the sections experience cyclic inelastic strains and should be checked as indicated in Sections 4.2.8 and 4.3.10.

If this is not the case, then no further cyclic checks are necessary.

A simple cyclic assessment criterion is thus to increment the environmental loading up to the required load level, and then reverse the environmental loads until they are fully removed from the structure and further applied to some 40% intensity in the opposite direction. If no yielding occurs under this reverse loading, the structure will not be susceptible to cyclic degradation, and no further cyclic assessment is necessary. If any members or joints yield under the reverse loading, a more detailed cyclic assessment should be performed.

An appropriate load history for assessment of cyclic storm loading is outlined in Section 6.7. The load history comprises an ordered load sequence representing the 10 000 year (10^{-4}) extreme storm loading (or the factored $\gamma_m \cdot \gamma_f$ 100-year load), combined with a number of cycles of the unfactored 100-year load from primary and opposing direction.

The 10 000 year storm waves are applied in decreasing sequence. Due to in-line current and generally higher wave crest than wave trough, the reverse load intensity is typically some 30% of the forward wave loading.

Comm. 5.4.3 Joint eccentricities

The nodes of the model of the framework should be the intersection points of the centre lines of legs, vertical diagonal members and plan braces. Offsets between the intersection points of brace member/chord centrelines at the joints, which are less the D/4 (D = the chord can diameter) and the corresponding member and eccentricities need not be included in the structural model.

Joint eccentricities will introduce additional end moments in the connected members. However, since non-linear pushover analyses are mainly used for global collapse and ultimate limit states assessment, the requirement for modelling of eccentricities should not exceed that of conventional ultimate limit state analyses.

Exceptions may be structures with large diameter legs and stocky member design. These may require special consideration. One modelling technique which has been used to represent the joint stiffness is to simulate chord stiffness between the intersection of the centrelines and the chord face as a rigid link with springs at the face representing the chord shell flexibility. Rigid links should not be used without also considering chord shell flexibility.

Joint flexibility reduces the rotational restraint of members in frames. and thereby reduces the buckling strength of the members. However, recent studies indicate that joint flexibility has minor influence on the collapse capacity of tubular frames /10/.

Comm. 5.4.4 Screening

At present the inclusion of joints generally complicates the modelling significantly, whatever software is being used. On this basis it is not practicable (and is anyway unnecessary) to model all joint characteristics explicitly. A practical (and instructive) approach is to begin with an initial analysis assuming all connections to be rigid and evaluating the utilisation of joints at peak load. A screening process can then be applied to identify all joints where the non-linear deformations and/or limiting capacity may influence the response. These can then be represented in the model and the analysis repeated to obtain a more realistic response prediction. The degree of utilisation beyond which non-linear deformations may significantly influence the local and global response cannot however be defined uniquely and definitive screening criteria cannot therefore be given. The screening level depends on the particular circumstances and joint type. Nevertheless a reasonable criterion is to include all joints where the utilisation exceeds 1.0 with partial resistance factors set to unity.

Comm. 5.4.5 Joint representation

Approach 1 relies only on readily available capacity formulae and can be implemented by way of simple springs at the brace-chord intersection. Additional steps are required to account for the interactions between forces/moments in the brace and chord. The redistribution of loads ad joints soften before reaching their peak capacity is neglected. The representation is identical whether the response characteristic is ductile or 'brittle' thus requiring separate consideration of validity limits.

Approach 2 provides a phenomenological representation of joint failure (typically introduced as springs) based on test data such that ductile and 'brittle' characteristics are accounted for, obviating the need for limiting strain criteria. Test data do not always represent the boundary constraints within a frame satisfactory and the validity of curve-fit representations needs to be considered before use. Additional steps are required to account for the interactions between forces/moments in the brace and chord.

Approach 3 accounts for co-acting loads, complex geometries and frame restraints but accuracy is dependent on careful calibration and convergence studies for the modelling strategy. Computational requirements can be onerous. Any potential for cracking and the consequent effects on flexibility and capacity are not modelled and must be addressed separately (eg with the use of strain criteria or explicit representation of cracks).

Comm. 5.4.7 Flexibility modelling

If initial elastic joint flexibility is to be taken into account, the true face-to-face length of the member should also be considered. Modelling tubular members with their face-to-face length instead of centre-centre length will increase the capacity of the member. Modelling joint flexibility will release some of the end restraints on the member , and thus reduce the capacity. In the study by /10/, face-to-face modelling with rigid joints increased the collapse capacity by 4-7%, compared to centre-centre modelling with rigid joints. Joints flexibility reduced the collapse capacity by 1-3% (compared to face-to-face modelling with rigid joints). Thus, modelling members with their face-to-face length and including joint flexibility lead to an increase in collapse capacity of 2-5% for the present case structures. However, the total effect of this detailed joint modelling is minor.

The investigations also indicated that the collapse load is fairly insensitive to variations in joint flexibility. For the present structures, joint (rotational) flexibility must be overestimated by a factor of two before the benefit of face-to-face member lengths are cancelled by reduced members end restraints (increase effective length factors).

Joint flexibility models developed by Holmås /29/, /30/, Fessler et al /19/ /20/ Efthymiou /31/, Buitrago et al /21/ and Chen et al /22/ have been compared and found to be in mutually good agreement, suggesting that they represent 'true' flexibility of the joint geometries in questions. See /10/.

This indicates that conventional (centre-centre/rigid joint) modelling gives conservative estimates of first member failure and collapse capacity for framed offshore structures. Face-to-face modelling with flexible joints gives some benefit (with respect to increased capacity estimates), but the benefit must be weighed against the increased complexity in modelling.

Comm. 8.2 Linear and non-linear analysis

There are three main sources for the safety margin inherent in the linear analyses:

- Member plastic bending capacity
- Simplification of frame stability with isolated column stability
- Redistribution of forces and moments

These effects are discussed in the following.

Plastic Bending Capacity

Most traditional methods limit the capacity to the point when the elastic analyses ceases to be valid. This is done also in cases where it is not a code requirement since until now non-linear methods have only been suitable for research work.

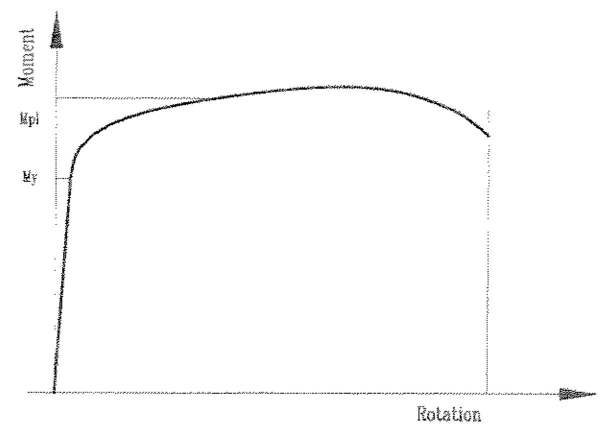


Figure 12-9 Plastic bending capacity of beams in bending

The difference stemming from the omission of the increased load carrying capacity due to the plastic capacity of the cross-section is dependent of type cross-section but is 1.27 for pipes. It should be noted that API RP 2A is partly accounting for this effect by use of a higher allowable stress for bending than axial stress.

Simplification of Frame Stability with Isolated Column Stability

The buckling failure of a structural system is in traditional methods carried out by calculating the stability of an isolated beam column which is given properties to best represent the stability capacity of the total frame. This is done by introduction of an effective length and equivalent uniform moment. Both simplifications are normally according to methods adding unintended safety to the structure.

In some cases the equivalent beam column concept may lead to an accurate prediction of the stability of a frame or truss structure, whereas in other cases it will result in considerable underestimation of the capacity of the structure.

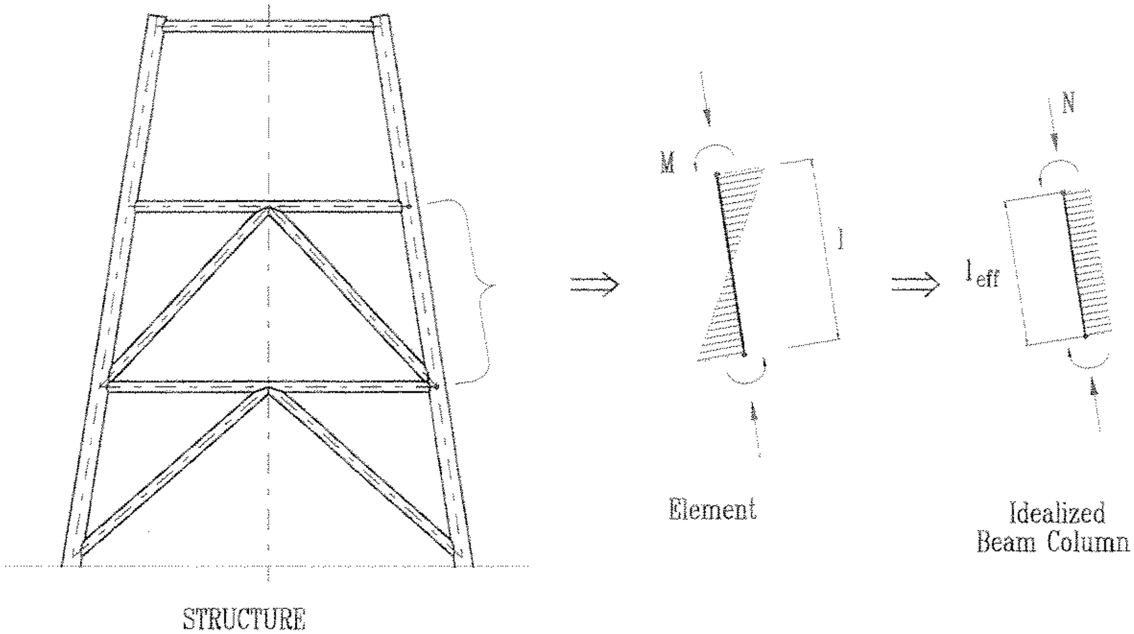


Figure 12-10 Isolated beam column

Separation of column stability and frame stability

The most sweeping simplification in current codes is the separation of column stability from frame stability. This has lead to quite simple column formulas and iterative procedures to determine frame stability. The effective length concept (*K*-factors) is introduced to incorporate frame effects into the column stability check. To simplify the design task further, the equivalent uniform moment concept (C_m-factors) is introduced to account for member P-δ effects.

The frame analysis and the code checking are separated. Member forces are determined through linear analysis of the frame system. Approximate formulae are used to estimate second-order forces, and the components' capacity and stability are evaluated by separate 'design equations'.

Effective length factors

The effective length concept separates the column stability from frame stability and reduces the design problem to that of an isolated member with given forces and end restraints. The "effective length" *KL* of a column defines part of the buckled deflection between points of zero curvature. I.e. the "effective length" is the length of a fictitious hinged-end column that would have the same Euler bifurcation load as the actual restrained column.

Since compression members fail through elasto-plastic buckling rather than Euler buckling, and since most beam-columns also carry some degree of transverse loading, use of the effective length concept is clearly an approximate procedure. Most codes accept the use of effective length factors determined through refined analysis of the beam-column in question. Such an analysis should consider the joint restraints, joint flexibility and joint sidesway. Furthermore, the joint must have sufficient capacity to actually impose the calculated end restraints at the required load level. In lieu of such refined analyses, most codes present recommended values for typical bracing configurations. These values are conservative, often based on an underlying assumption of a fully optimised truss work where buckling stresses in compression members and yield stresses in tension members are reached at the same level of live load. See /28/. On this basis, no restraint would be supplied at the joint, and the effective length factor is selected as for a member with pinned end(s).

Choice of effective length factors has significant impact on the design strength estimated by conventional procedures. Good agreement between code stability formulae and non-linear pushover analyses are found if effective length factors are determined by refined analyses (*reference*).

Equivalent uniform moment, C_m

The equivalent uniform moment concept is an approximate measure to introduce second-order effects in the design. Instead of performing non-linear analyses to determine the second-order forces in each member, bending moments from linear analyses are multiplied by $C_m / (1 - N / P_{Ek})$ to estimate the 'real' first order + second order moments.

Thus, the concept of equivalent moment replaces the design of a beam-column subject to an arbitrary combination of end moments with the design of an equivalent beam column subject to equal and opposite end moments of magnitude $M_{eq} = C_m M_B$.

The C_m factor is easily determined by simple formulae for some loading and support conditions. For most conditions however, accurate formulae become quite elaborate and actual design is carried out with conservative approximations.

Redistribution of forces and Moments

In traditional methods the beneficial effect that forces may shed from the heavy loaded parts of the structure to less loaded parts is neglected. In non-linear methods these effect are automatically taken care of and will in many cases offering considerable additional capacity

It should be recognised that the structural codes are the results of a historical development, reflecting both the state of knowledge and the state of engineering 'tools' during each period. The codes ensure an adequate standard of safety of the resulting structures and work as a guidance for the designer as to which approaches might be accepted for design and construction. As such, it is not a requirement for the codes to present correct, theoretical solutions for a design, but rather to present reasonably consistent and rational procedures for the creation of real structures with the analysis tools generally available. Current design equations are developed for hand-calculations, which require very simple formulae.

Comm. 8.3 LRFD format

The definition of ultimate limit state is fairly similar in different codes which are making use of this expression. The following definitions are given in Eurocode 3:

"Limit states:

Limit states are states beyond which the structure no longer satisfies the design performance requirements.

Limit states are classified into:

- ultimate limit states
- serviceability limit states

Ultimate limit states are those associated with collapse, or with other forms of structural failure which may endanger the safety of people.

States prior to structural collapse which, for simplicity, are considered in place of the collapse itself are also classified and treated as ultimate limit states.

Ultimate limit states which may require consideration include:

- *Loss of equilibrium of the structure or any part of it, considered as a rigid body,*
- *Failure by excessive deformation, rupture, or loss of the structure or any part of it, including supports and foundations."*

Most limit state design codes prescribe different values for the load factors for different type of loads depending of the uncertainty associated with the load. The codes also prescribes that two or more combinations of the loads need to be checked and that the load factors are dependent upon the reduced probability of non-correlated loads to reach their maximum at the same time. The resulting probability of failure is illustrated in Figure 12-11 as values obtained if NPD safety format is used for some typical assumptions for the load and resistance statistical variation. As the failure, probabilities shown in this figure are extremly sensitive to the assumptions made it should only be read in a relative sense for the purpose of making comparisons.

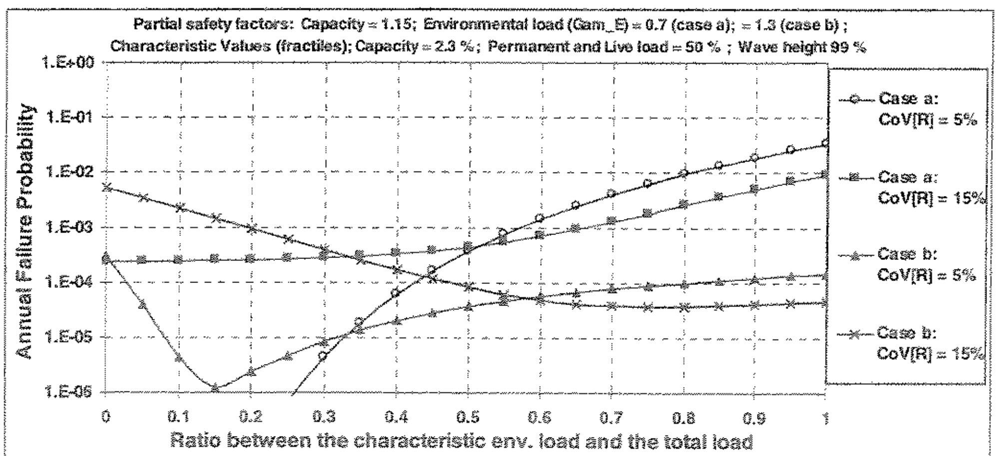


Figure 12-11 Annual probability of failure for structures designed according to NPD safety format.

Comm. 8.4 API WSD format

The use of a single safety factor as the RSR which only apply to the environmental loads will inevitably imply variable probability of failure. This is illustrated in Figure 12-12 showing that the RSR safety format is giving consistent reliability level only for structures where the environmental loads dominate. The figure should only be read in a relevant manner, as the values for probabilities of failure are extremely sensitive to the assumptions made.

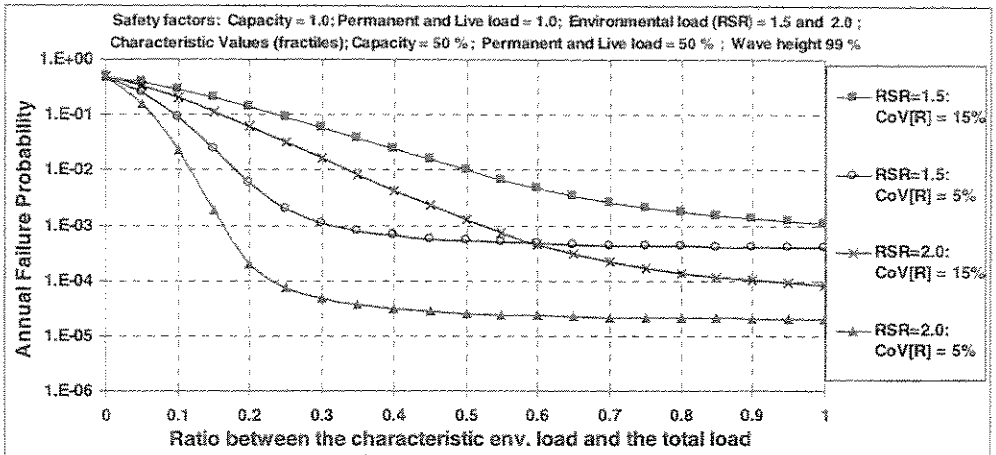


Figure 12-12 Annual probability of failure for structures designed according to API RSR format.

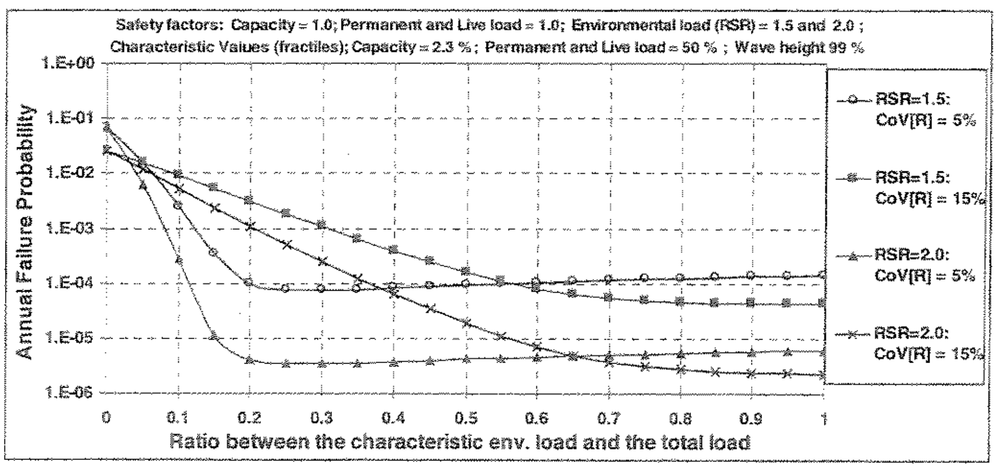


Figure 12-13 Annual probability of failure for structures designed according to API RSR safety format but with lower bound characteristic resistance values. (2.3% fractile)

The reserve strength ratio (RSR) is in the API code defined with the best estimate in contrast to lower bound as the characteristic value for resistance. This is a less suitable characteristic value in order to obtain consistent structural reliability for different failure modes. In most codes, a low percentile characteristic value (lower bound) is prescribed for establishing the design resistance. Guidance on how to determine the expected values for different failure modes are also less available in engineering hand- and textbooks. In Figure 12-13 the resulting probability of failure is shown if a low characteristic value for the resistance is used. By comparison with Figure 12-12 it can be seen that use of a low fractile for the characteristic yields more uniform reliability level for different uncertainties in the failure modes. Two variations of the resistance 5% and 15% are shown. The first value corresponds to the variation for the yield stress while the second value may be representative for joint strength.

It is also important to note that the reliability level is significantly affected by the selection of characteristic values.

- oOo -

13 Examples

13.1 North Sea Jacket analysed with ABAQUS

13.1.1 Introduction

This section describes the work undertaken for the ultimate strength analysis of a typical modern North Seas jacket structure.

13.1.2 Description

The computer model includes the topside, the jacket and the foundation. Figure 13-1 shows the geometry of the structure. The basic analysis method adopted consists of two steps The first analysis step models the self weight and the topside load with a factor 1.15 to account for their uncertainty. In the second step the 100-year return period storm loads (wave and wind) are applied and monotonically increased until the limit load is reached. Then the post limit load behaviour is obtained using an arc length load/ displacement control method.

13.1.3 Element model

Each structural member is modelled with a number of three-node quadratic beam elements, designated B32 in the ABAQUS element library. This member model refinement is essential for two reasons: (a) sufficient points along the member are needed to ensure the accurate location of the position at which any plastic hinge may form and (b) to ensure that column buckling behaviour is adequately represented.

The materials used for this structure were classified as either Grade 355 or Grade 450 steel according to BS7191. Isotropic strain hardening is assumed for all the material used.

The piled foundation was modelled in detail as the capacity of the piles at ultimate strength may significantly influence the capacity of the system. Three-node beam elements were used to model the piles, while non-linear springs were used to model P-y, T-z and Q-z curves. The soil springs are attached to the beam nodal point with each spring representing an appropriate contributory length of the pile.

13.1.4 Load model

The operational loads corresponding to still-water case in the design analysis software are translated into the required ABAQUS format. The self-weight and buoyancy load generated in ABAQUS.

The 100 year environmental loads - which contain loads from wind, waves and current - that produced the greatest overturning moment and base shear in the structure were used. The loads do not contain any load factors except for the Dynamic Amplification Factor (DAF).

13.1.5 Analysis

A series of non-linear analyses are performed to determine the Reserve Strength Ratio (RSR). The loads are applied in two main steps. In the first step 1.15 times of all operational loads and self-weight are applied. In the next step lateral loads corresponding to a 100-year storm are applied. These loads are monotonically increased until collapse of the structure is detected.

The 100-year storm load conditions from three different directions were used in the analyses. Three configurations of the structure were modelled. The structure was analysed a) without imperfections, b) with imperfections in heavily utilised members, modelled as a parabolic curve with a maximum deflection of 2/1000 of the member length at the centre c) with joint flexibility included at highly utilised joints. Elements connecting the member to the joint were replaced with non-linear springs.

13.1.6 Results

The displacement of a node in the centre of the structure was monitored in order to capture the global load-displacement response. This is shown for each of the loading conditions in Figure 13-2 to Figure 13-4. The post collapse behaviour of the structure was captured using the Riks analysis method.

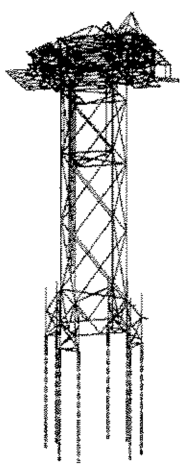


Figure 13-1 Geometry of a typical North Sea structure.

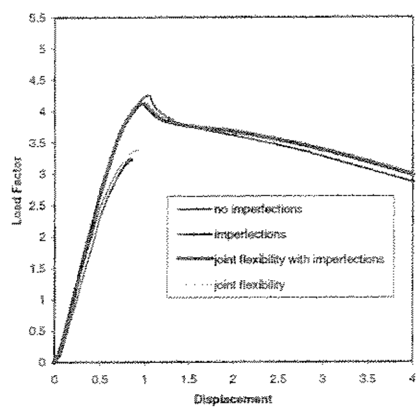


Figure 13-2 Load-displacement curve for the northerly load condition

The displaced shapes of the structure loaded from the north at the end of the analyses are shown in Figure 13-5.

The peak load in the structure loaded from the north is reached after a global displacement of 1.1m in the model without imperfections as opposed to a displacement of 0.9m in the model with imperfections. Plastic hinges have formed in the diagonal bracing in the mid section of the structure. Plastic strains at this point are in the region of 2 - 4%. Plastic hinges have also formed at the bracing connecting the pile sleeve to the jacket.

Modelling flexibility at selected highly utilised joints, detailed in Figure 13-8, had the effect of softening the structure's overall stiffness and further reduced the RSR. The RSR with joint flexibility included was reduced by approximately 20% for both the unperturbed structure and the structure with imperfections included.

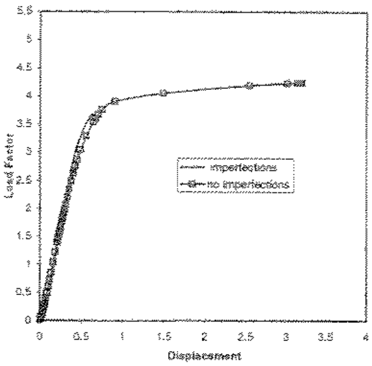


Figure 13-3 Load-displacement curve for the north-westerly load condition

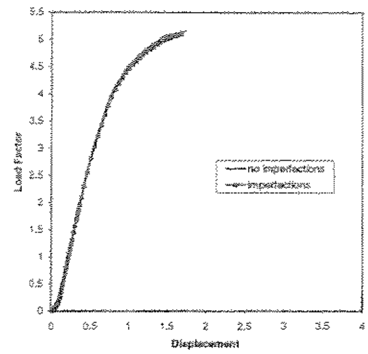
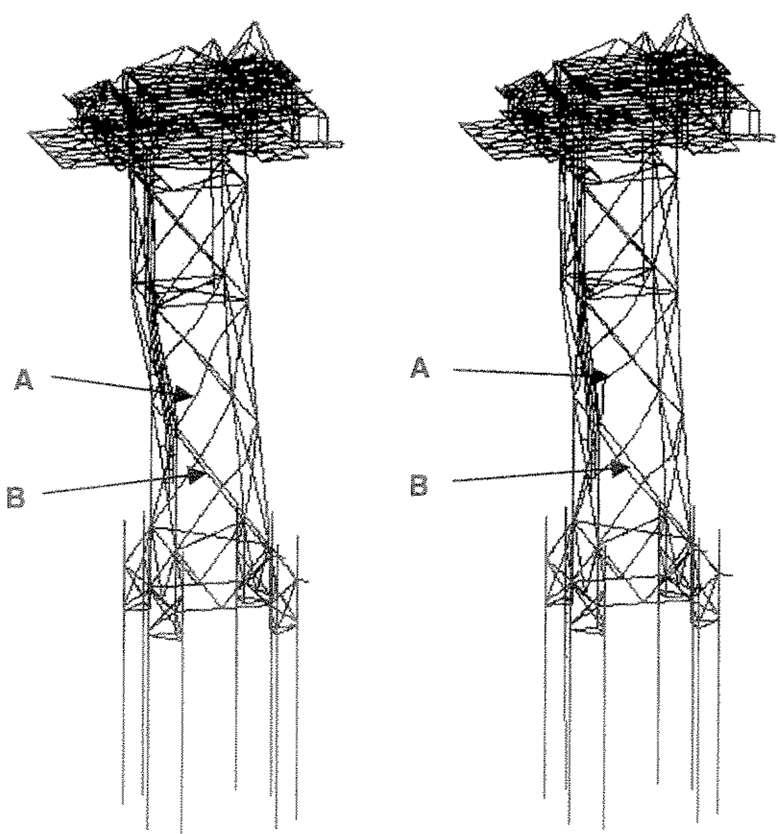


Figure 13-4 Load-displacement curve for the westerly load condition.



(a) (b)

Figure 13-5 Displaced shape of the structure loaded from the north, (a) without imperfections and (b) with imperfections in highly utilised bracing members.

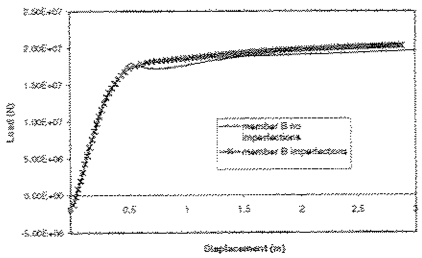


Figure 13-6 Local P-δ behaviour for bracing, member B, in tension.

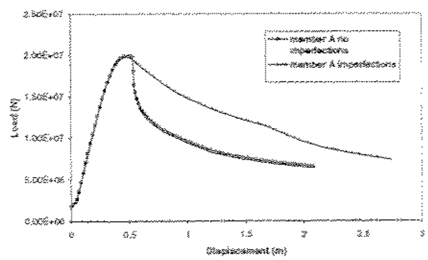


Figure 13-7 Local P- δ behaviour for bracing, member A, in compression.

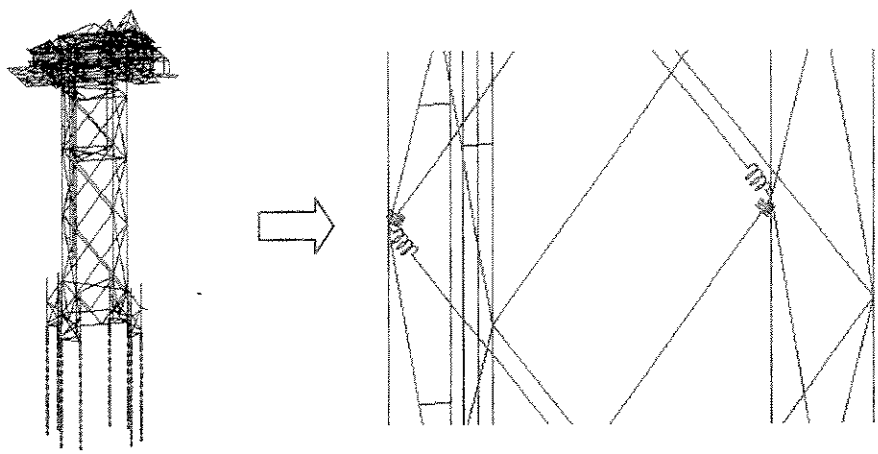


Figure 13-8 Geometry of the joint flexibility modelled as non-linear springs at selected joints.

13.2 North Sea jacket analysed with USFOS.

13.2.1 Introduction

An ultimate strength non-linear analysis for a realistic jacket is performed.

The ultimate strength is found by use of the program USFOS, which is representing a Non-linear beam-column analytical technique.

USFOS operates on element stress resultants, i.e. forces and moments. Material non-linearities are modelled by plastic hinges at element midspan , and at element ends.

The basic element formulation in USFOS is based on the exact solution of the differential equation subjected to end forces.

13.2.2 Description

The platform is a 4-legged X-braced riser jacket situated in the North-Sea. The platform was installed in 1994 at a water depth of 70.1 m. The platform is supported with 4 insert piles. The platform has a total of four risers.

Weight of topside including flaretower is 3268 Tonnes.

13.2.3 Element model

The model is based on an existing model generated in SESAM input format. The original model includes detailed modelling of topside and flaretower. For the purpose of analysing the jacket, the topside is simplified, and the flare tower is removed (loads from flaretower is included).

Number of elements	:	608
Number of nodes	:	382
material model	:	elasto-plastic with strain hardening
Joint representation	:	rigid connections at brace/chord intersection
Joint representation	:	rigid connections at brace/chord intersection
		(Generally the member cross-sections are increased at the joints, and therefore the capacity of the joints are higher than for the members. A test analysis shows that the global capacity was not influenced by including joint capacity check according to API.)
Initial deflection, e/L	:	Leg members: 0.002, Diagonal Braces: 0.003. The imperfections are assigned in a pattern sympathetic to the collapse mode.

13.2.4 Load model

A combination of selfweight and buoyancy is applied to a loadlevel of 1.15 (= 1.0 * 1.15). The factor 1.15 accounting for material factor.

The environmental loads are then gradually increased until collapse of the platform. The environmental loads are combinations of wind, wave and current, wind and wave with 100-year recurrence period, current with 10 years.

The platform is checked for eight different environmental directions.

Hydrostatic pressure is calculated on assigned elements. The hydrostatic pressure is calculated and imposed on the element at the initiation of the analysis. By including hydrostatic pressure in the USFOS analysis, the plastic axial and moment capacities of the section is affected.

13.2.5 Analysis

The USFOS non-linear analysis has followed the following basic procedure:

- The load is applied in steps
- The nodal coordinates are updated after each load step

- The structure stiffness is assembled at each load step. The element stiffnesses are then calculated from the updated geometry.
- At every load step each element is checked to see whether the forces exceed the plastic capacity of the cross section. If such an event occurs, the load step is scaled to make the forces comply "exactly" with the yield condition.
- A plastic hinge is inserted when the element forces have reached the yield surface. The hinge is removed if the element later is unloaded and becomes elastic.
- The load step is reversed (the load is reduced) if global instability is detected.

The load has been applied incrementally. The size of the increments have been varied so that large steps have been prescribed in the linear range, and smaller steps with increasingly non-linear behaviour.

13.2.6 Results

The jacket has been checked for environmental loads (wind, wave, and current) from 8 directions.

North and Northwest directions are the directions with highest resulting basic loads, and are therefore the directions gaining lowest loadlevel before failure. The discussion of the results will therefore be concentrated on these two directions.

Table 13-1 Results from Non-linear analysis

		Resulting basic load		USFOS				
Load combination	No.	Wind (MN)	Wave (MN)	1st member plast. Member/Loadlevel		Member initiating collapse Member/Loadlevel		Max Loadlevel
West	2	3.31	17.81	Not checked		**1109**	4.016	4.063
Southwest	3	3.43	12.53	Not checked		***406***	*6.022*	*6.114*
South	4	3.60	15.46	Not checked		**1409**	4.214	4.260
Southeast	5	3.33	15.00	Not checked		***305***	*5.400*	*5.466*
East	6	3.12	10.07	Not checked		**1108**	5.760	5.854
Northeast	7	3.09	10.50	Not checked		***105***	*6.450*	*6.724*
North	8	3.34	24.26	After max. load		**1408**	**2.960**	2.974
Northwest	9	3.30	24.16	*1107*	*3.493*	***205***	*3.833*	*3.848*

The maximum environmental load multiplier is 2.96.

With environmental wind from north, member 1408 (diagonal brace) buckles at loadlevel 2.96. This event is initiating global collapse. No member has experienced full plasticity at this stage, thus the limit of 5% nominal tension strain is not exceeded.

<u>General for diagonal load direction:</u>
The collapse is initiated by buckling in opposite leg member, between elevation –44.0m and –23.0m. This is the case for all diagonal load directions (see Figure 13-9).

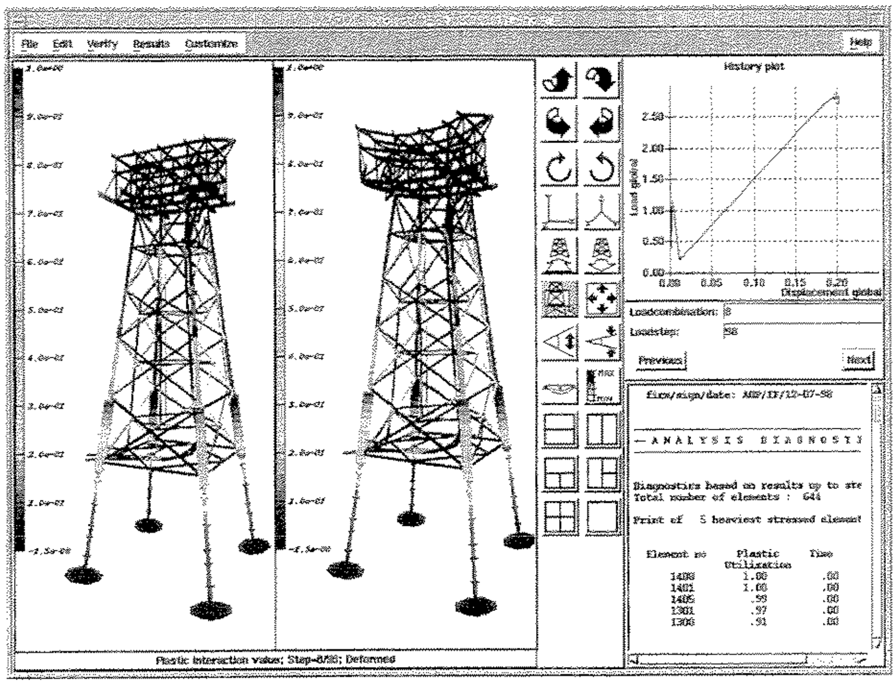


Figure 13-9   Deformed structure at loadlevels described in table 4.1, Loads from North (deformation are scaled to 10 times actual value)

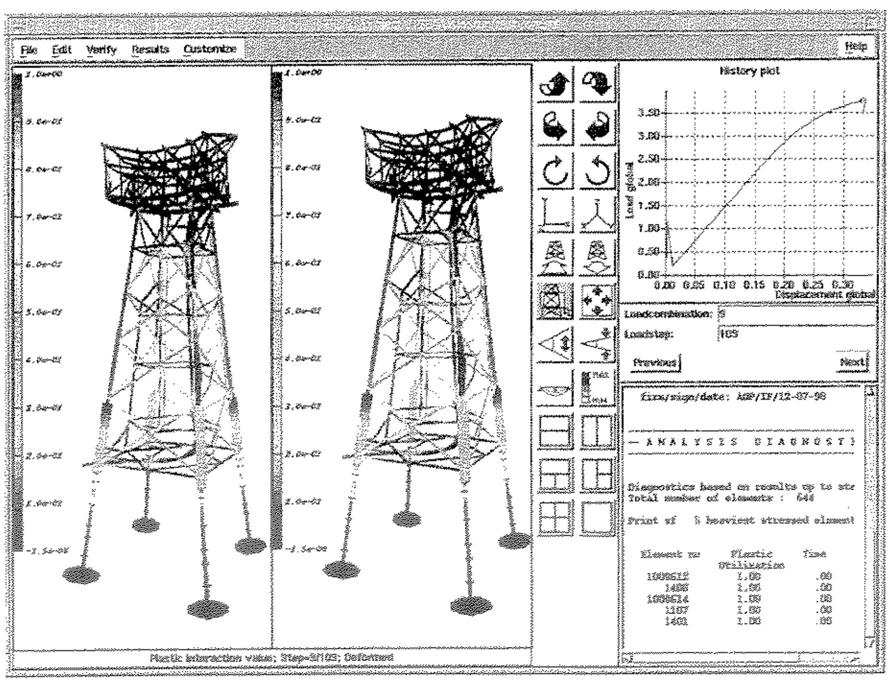


Figure 13-10   Deformed structure at loadlevels described in table 4.1, Loads from North West (deformation are scaled to 10 times actual value)

General for head-on load direction:
The collapse is initiated by buckling of diagonal braces in panels parallel to load direction (see Figure 13-10).

Comparison with traditional analysis
A traditional analysis with the same boundary conditions, has not been performed. To give an idea of the result of an elastic analysis, the loadlevel of first yield gives a good indication of the capacity.

Table 13-2 Comparison with traditional analysis

	Trad. analysis	Non-linear anal.	Increase in loadlevel
Loadlevel, Loads from North:	1.67	2.96	77 % (2.96/1.67 = 1.77)

- o0o -