

Design of steel structures

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Foreword

The NORSOK standards are developed by the Norwegian petroleum industry to ensure adequate safety, value adding and cost effectiveness for petroleum industry developments and operations. Furthermore, NORSOK standards are, as far as possible, intended to replace oil company specifications and serve as references in the authorities' regulations.

The NORSOK standards are normally based on recognised international standards, adding the provisions deemed necessary to fill the broad needs of the Norwegian petroleum industry. Where relevant, NORSOK standards will be used to provide the Norwegian industry input to the international standardisation process. Subject to development and publication of international standards, the relevant NORSOK standard will be withdrawn.

The NORSOK standards are developed according to the consensus principle generally applicable for most standards work and according to established procedures defined in NORSOK A-001.

The NORSOK standards are prepared and published with support by The Norwegian Oil Industry Association (OLF) and Federation of Norwegian Manufacturing Industries (TBL).

NORSOK standards are administered and published by Standards Norway.

Annex A, K, L, M and N are normative parts of this NORSOK standard.

Introduction

This NORSOK standard is intended to fulfil PSA regulations relating to design and outfitting of facilities etc. in the petroleum activities.

The design principles follow the requirements in ISO 19900.

1 SCOPE

This NORSOK standard specifies guidelines and requirements for design and documentation of offshore steel structures.

This NORSOK standard is applicable to all type of offshore structures made of steel with a specified minimum yield strength less or equal to 500 MPa. For steel with higher characteristic yield strength, see Clause 12.

2 NORMATIVE AND INFORMATIVE REFERENCES

The following standards include provisions and guidelines which, through reference in this text, constitute provisions and guidelines of this NORSOK standard. Latest issue of the references shall be used unless otherwise agreed. Other recognized standards may be used provided it can be shown that they meet or exceed the requirements and guidelines of the standards referenced below.

2.1 Normative references

API RP 2T,	Planning, Designing and Constructing Tension Leg Platforms
DNV,	Rules for classification of ships
DNV,	Classification Note 30.7
DNV-OS-C101,	Design of offshore Steel Structures
DNV-RP-C201,	Buckling Strength of Plated Structures
DNV-RP-C203,	Fatigue Strength Analysis of Offshore Steel Structures
DNV-RP-C202,	Buckling of Shells
DNV-OS-C502,	Offshore Concrete Structures
ISO 19900,	Petroleum and natural gas industries – General requirements for offshore structures
ISO 19902,	Petroleum and natural gas industries – Fixed steel offshore structures
NORSOK G-001,	Soil investigation
NORSOK M-001,	Materials selection
NORSOK M-101,	Structural steel fabrication
NORSOK M-120,	Material data sheets for structural steel
NORSOK N-001,	Structural design
NORSOK N-003,	Action and action effects
NORSOK N-006,	Assessment of structural integrity for existing offshore load-bearing structures
NORSOK Z-001,	Documentation for operation
NS 3481,	Soil investigation and geotechnical design for marine structures
NS-EN 1993-1-1,	Eurocode 3: Design of steel structures - Part 1-1: General rules and rules for buildings
NS-EN 1993-1-5,	Eurocode 3: Design of steel structures - Part 1-5: Plated structural elements
NS-EN 1993-1-8,	Eurocode 3: Design of steel structures - Part 1-8: Design of joints
VMO Standards	As defined in DNV-OS-H101, Marine Operations, General

2.2 Informative references

See Clause 11.

3 TERMS, DEFINITIONS, ABBREVIATIONS AND SYMBOLS

For the purposes of this NORSOK standard, the following terms, definitions and abbreviations apply.

3.1 Terms and definitions

3.1.1

can

verbal form used for statements of possibility and capability, whether material, physical or casual

3.1.2

may

verbal form used to indicate a course of action permissible within the limits of the standard

3.1.3

Norwegian petroleum activities

petroleum activities where Norwegian regulations apply

3.1.4

operator

company or an association which through the granting of a production licence is responsible for the day to day activities carried out in accordance with the licence

3.1.5

petroleum activities

offshore drilling, production, treatment and storage of hydrocarbons

3.1.6

principal standard

standard with higher priority than other similar standards

NOTE Similar standards may be used as supplements, but not as alternatives to the principal standard.

3.1.7

shall

verbal form used to indicate requirements strictly to be followed in order to conform to the standard and from which no deviation is permitted, unless accepted by all involved parties

3.1.8

should

verbal form used to indicate that among several possibilities one is recommended as particularly suitable, without mentioning or excluding others, or that a certain course of action is preferred but not necessarily required

3.2 Abbreviations

ALS	accidental limit state
API	American Petroleum Institute
BSI	British Standards Institution
CTOD	crack tip opening displacement
DAF	dynamic amplification factor
DC	design class
DFF	design fatigue factor
DFI	design, fabrication and installation
DNV	Det Norske Veritas
EA	environmental actions
ECCS	European Convention for Constructional Steelwork
FE	finite element
FEM	finite element method
FLS	fatigue limit state
FPSO	floating production storage and offloading
HF	high frequency
IMO	International Maritime Organisation
ISO	International Organisation for Standardisation
LF	low frequency
MDS	material data sheet
MPI	magnetic particle inspection
NDT	non-destructive testing
NMD	Norwegian Maritime Directorate
NPD	Norwegian Petroleum Directorate
RAO	response amplitude operator
RCS	Recognised Classification Society
ROV	remotely operated vehicle

SDOF	single degree of freedom
SQL	steel quality level
SCF	stress concentration factor
TLP	tension leg platform
ULS	ultimate limit state
WF	wave frequency

3.3 Symbols

A	cross sectional area, accidental action, parameter, full area of the brace/chord intersection
A_c	cross sectional area of composite ring section
A_e	effective area
A_w	cross sectional area of web
B	hoop buckling factor
C	rotational stiffness, factor
C_e	critical elastic buckling coefficient
C_h	elastic hoop buckling strength factor
C_m	reduction factor
C_{my}, C_{mz}	reduction factor corresponding to the member y and z axis respectively
C_0	factor
D	outer diameter of chord, cylinder diameter at a conical transition, outside diameter
D_c	diameter to centroid of composite ring section
D_e	equivalent diameter
D_j	diameter at junction
D_{max}	maximum measured diameter
D_{min}	minimum measured diameter
D_{nom}	nominal diameter
D_s	outer cone diameter
E	Young's modulus of elasticity, $2.1 \cdot 10^5$ MPa
G	shear modulus
I, I_z, I_s	moment of inertia
I_c	moment of inertia for ring composite section
I_{ch}	moment of inertia
I_{cT}	moment of inertia of composite ring section with external hydrostatic pressure and with effective width of flange
I_e	effective moment of inertia
I_p	polar moment of inertia
K_a	factor
L	length, distance
L_1	distance to first stiffening ring in tubular section
L_c	distance to first stiffening ring in cone section along cone axis
L_r	ring spacing
$M_{pl,Rd}$	design plastic bending moment resistance
M_{Rd}	design bending moment resistance
$M_{red,Rd}$	reduced design bending moment resistance due to torsional moment
M_{Sd}	design bending moment
$M_{T,Sd}$	design torsional moment
$M_{T,Rd}$	design torsional moment resistance
$M_{y,Rd}$	design in-plane bending moment resistance
$M_{y,Sd}$	in-plane design bending moment
$M_{z,Rd}$	design out-of-plane bending moment resistance
$M_{z,Sd}$	out-of-plane design bending moment
$N_{c,Rd}$	design axial compressive resistance
$N_{can,Rd}$	design axial resistance of can
$N_{cg,Rd}$	axial compression resistance of a grouted, composite member
N_{cl}	characteristic local buckling resistance
N_{cl}	characteristic local buckling resistance
$N_{cl,Rd}$	design local buckling resistance
N_E	Euler buckling strength
$N_{E,dent}$	Euler buckling strength of a dented tubular member, for buckling in-line with the dent
N_{Eg}	elastic Euler buckling load of a grouted, composite member

N_{Ey}, N_{Ez}	Euler buckling resistance corresponding to the member y and z axis respectively
N_{Sd}	design axial force
$N_{t,Rd}$	design axial tension resistance
$N_{x,Rd}$	design axial resistance in x direction
$N_{x,Sd}$	design axial force in x direction
$N_{y,Rd}$	design axial resistance in y direction
$N_{y,Sd}$	design axial force in y direction
P_{Sd}	design lateral force
Q	factor
Q_f	chord action factor
Q_g	chord gap factor
Q_u	strength factor
Q_{yy}	angle correction factor
Q_β	geometric factor, tubular joint geometry factor
R	radius, radius of chord, radius of conical transition
R_d	design resistance
R_k	characteristic resistance
S_d	design action effect
S_k	characteristic action effect
T	thickness of tubular sections, thickness of chord
T_c	chord-can thickness
T_n	nominal chord member thickness
V_{Rd}	design shear resistance
V_{Sd}	design shear force
W	elastic section modulus
Z	plastic section modulus
a	stiffener length, crack depth, weld throat, factor
a_i	initial crack depth
a_m	maximum allowable defect size
a_u	calculated crack depth at unstable fracture
b	factor
b_e	effective width
c	factor
d	diameter of tubular cross section, brace diameter
d_0, d_1, d_2	distance
e_f	flange eccentricity
f_c	characteristic axial compressive strength
$f_{ch,Rd}$	design axial compressive strength in the presence of hydrostatic pressure
f_{cj}	corresponding tubular or cone characteristic compressive strength
f_{cl}	characteristic local buckling strength
$f_{cl,Rd}$	design local buckling strength, design local buckling strength of undamaged cylinder
f_{clc}	local buckling strength of conical transition
f_{cle}	characteristic elastic local buckling strength
f_{cr}	critical buckling strength
f_d	design yield strength
f_E	Euler buckling strength
f_{Epx}, f_{Epy}	Euler buckling strength corresponding to the member y and z axis respectively
f_{Ey}, f_{Ez}	Euler buckling strength corresponding to the member y and z axis respectively
f_h	characteristic hoop buckling strength
f_{he}	elastic hoop buckling strength for tubular section
f_{hec}	elastic hoop buckling strength for cone section
$f_{h,Rd}$	design hoop buckling strength
$f_k, f_{kx}, f_{ky}, f_{kp}$	characteristic buckling strength
f_m	characteristic bending strength
$f_{m,Rd}$	design bending strength
$f_{m,Red}$	reduced bending strength due to torsional moment
$f_{mh,Rd}$	design bending resistance in the presence of external hydrostatic pressure
$f_{th,Rd}$	design axial tensile resistance in the presence of external hydrostatic pressure
f_y	characteristic yield strength
$f_{y,b}$	characteristic yield strength of brace
$f_{y,c}$	characteristic yield strength of chord

g	gap
h	height
i	radius of gyration
i_e	effective radius of gyration
k, k_g, k_l, k_σ	buckling factor
l, l_L	length, element length
l_e	effective length
p_{Sd}	design hydrostatic pressure
r	radius, factor
t	thickness
t_c	cone thickness
t_{eff}	effective thickness of chord and internal pipe of a grouted member
α	coefficient, angle between cylinder and cone geometrical coefficient, factor
β	factor
γ	factor
γ_{BC}	additional building code material factor
γ_f	partial factor for actions
γ_M	resulting material factor
γ_{M0}	material factor for use with EN-1993-1-1
γ_{M1}	material factor for use with EN-1993-1-1
γ_{M2}	material factor for use with EN-1993-1-1 and EN-1993-1-8
ε	factor
η	hoop buckling factor
θ	angle
θ_c	the included angle for the compression brace
θ_t	the included angle for the tension brace
$\bar{\lambda}$	reduced slenderness, column slenderness parameter
$\bar{\lambda}_e$	reduced equivalent slenderness
$\bar{\lambda}_s$	reduced slenderness, shell slenderness parameter
μ	coefficient, geometric parameter
ν	Poisson's ratio
ρ	rotational stiffness factor
$\sigma_{a,Sd}$	design axial stress in member
$\sigma_{ac,Sd}$	design axial stress including the effect of the hydrostatic capped end axial stress
$\sigma_{at,Sd}$	design axial stress in tubular section at junction due to global actions
$\sigma_{equ,Sd}$	equivalent design axial stress within the conical transition
$\sigma_{h,Sd}$	design hoop stress due to the external hydrostatic pressure
$\sigma_{hc,Sd}$	design hoop stress at unstiffened tubular-cone junctions due to unbalanced radial line forces
$\sigma_{hj,Sd}$	net design hoop stress at a tubular-cone junction
σ_j,Sd	design von Mises' equivalent stress
$\sigma_{m,Sd}$	design bending stress
$\sigma_{mc,Sd}$	design bending stress at the section within the cone due to global actions
$\sigma_{mlc,Sd}$	local design bending stress at the cone side of unstiffened tubular-cone junctions
$\sigma_{mlt,Sd}$	local design bending stress at the tubular side of unstiffened tubular-cone junctions
$\sigma_{mt,Sd}$	design bending stress in tubular section at junction due to global actions
$\sigma_{my,Sd}$	design bending stress due to in-plane bending
$\sigma_{mz,Sd}$	design bending stress due to out-of-plane bending
$\sigma_{p,Sd}$	design hoop stress due to hydrostatic pressure
$\sigma_{tot,Sd}$	total design stress
$\sigma_{q,Sd}$	capped end axial design compression stress due to external hydrostatic pressure
$\tau_{T,Sd}$	shear stress due to design torsional moment
ψ, ψ_x, ψ_y	factors
ϕ	factor

4 GENERAL PROVISIONS

All relevant failure modes for the structure shall be identified and it shall be checked that no corresponding limit state is exceeded.

In this NORSOK standard the limit states are grouped into:

1. Serviceability limit states
2. Ultimate limit states
3. Fatigue limit states
4. Accidental limit states

For definition of the groups of limit states, reference is made to ISO 19900.

The different groups of limit states are addressed in designated clauses of this NORSOK standard. In general, the design needs to be checked for all groups of limit states.

The general safety format may be expressed as:

$$S_d \leq R_d \quad (4.1)$$

where

S_d	=	$S_k \gamma_f$	Design action effect
R_d	=	$\frac{R_k}{\gamma_M}$	Design resistance
S_k	=		Characteristic action effect
γ_f	=		partial factor for actions
R_k	=		Characteristic resistance
γ_M	=		Material factor

In this NORSOK standard the values of the resulting material factor are given in the respective clauses.

General requirements for structures are given in NORSOK N-001.

Determination of actions and action effects shall be according to NORSOK N-003.

The steel fabrication shall be according to the requirements in NORSOK M-101.

5 STEEL MATERIAL SELECTION AND REQUIREMENTS FOR NON-DESTRUCTIVE TESTING

5.1 Design class

Selection of steel quality and requirements for inspection of welds shall be based on a systematic classification of welded joints according to the structural significance and complexity of joints. The main criterion for decision of DC of welded joints is the significance with respect to consequences of failure of the joint. In addition the geometrical complexity of the joint will influence the DC selection.

The selection of joint design class shall be in compliance with Table 5-1 for all permanent structural elements. A similar classification may also be used for temporary structures.

Table 5-1 Classification of structural joints and components

Design Class ¹⁾	Joint complexity ²⁾	Consequences of failure
DC1	High	Applicable for joints and members where failure will have substantial consequences ³⁾ and the structure possesses limited residual strength. ⁴⁾
DC2	Low	
DC3	High	Applicable for joints and members where failure will be without substantial consequences ³⁾ due to residual strength. ⁴⁾
DC4	Low	
DC5	Any	Applicable for joints and members where failure will be without substantial consequences. ³⁾

Notes:

- 1) Guidance for classification can be found in Annex K, L, M and N.
- 2) High joint complexity means joints where the geometry of connected elements and weld type leads to high restraint and to triaxial stress pattern. E.g., typically multiplanar plated connections with full penetration welds.
- 3) "Substantial consequences" in this context means that failure of the joint or member will entail
 - danger of loss of human life;
 - significant pollution;
 - major financial consequences.
- 4) Residual strength means that the structure meets requirements corresponding to the damaged condition in the check for accidental damage limit states, with failure in the actual joint or component as the defined damage. See Clause 12.

5.2 Steel quality level

Selection of steel quality level for a structural component shall normally be based on the most stringent DC of joints involving the component. Through-thickness stresses shall be assessed.

The minimum requirements for selection of steel material are found in Table 5-2. Selection of a better steel quality than the minimum required in design shall not lead to more stringent requirements in fabrication.

The principal standard for specification of steels is NORSOK M-120, Material data sheets for rolled structural steel. Material selection in compliance with NORSOK M-120 assures toughness and weldability for structures with an operating temperature down to -14 °C.

Cast and forged steels shall be in accordance with recognised standards.

If steels of higher specified minimum yield strength than 500 MPa or greater thickness than 150 mm are selected, the feasibility of such a selection shall be assessed in each case.

Traceability of materials shall be in accordance with NORSOK Z-001.

Table 5-2 Correlation between design classes and steel quality level

Design Class	Steel Quality Level			
	I	II	III	IV
DC1	X			
DC2	(X)	X		
DC3	(X)	X		
DC4	(X)		X	
DC5				X

(X) = Selection where the joint strength is based on transference of tensile stresses in the through thickness direction of the plate. See Clause 12.

5.3 Welding and non-destructive testing (NDT)

The required fracture toughness level shall be determined at the design stage for joints with plate thickness above 50 mm when for joints in design class DC1, DC2 or DC3.

The extent of non-destructive examination during fabrication of structural joints shall be in compliance with the inspection category. The selection of inspection category for each welded joint shall be in accordance with Table 5-3 and Table 5-4 for joints with low and high fatigue utilisation respectively.

Welds in joints below 150 m water depth should be assumed inaccessible for in-service inspection.

The principal standard for welder and welding qualification, welding performance and non-destructive testing is NORSOK M-101.

Table 5-3 Determination of inspection category for details with low fatigue utilisation ¹⁾

Design Class	Type of and level of stress and direction in relation to welded joint.	Inspection category ²⁾
DC1 and DC2	Welds subjected to high tensile stresses transverse to the weld. ⁶⁾	A
	Welds with moderate tensile stresses transverse to the weld and/or high shear stresses. ⁷⁾	B ³⁾
	Welds with low tensile stresses transverse to the weld and/or moderate shear stress. ⁸⁾	C ⁴⁾
DC3 and DC4	Welds subjected to high tensile stresses transverse to the weld. ⁶⁾	B ³⁾
	Welds with moderate tensile stresses transverse to the weld and/or high shear stresses. ⁷⁾	C ⁴⁾
	Welds with low tensile stresses transverse to the weld and/or moderate shear stress. ⁸⁾	D ⁵⁾
DC5	All load bearing connections.	D
	Non load bearing connections.	E

Notes:

- 1) Low fatigue utilisation means connections with calculated fatigue life longer than 3 times the required fatigue life (design fatigue life multiplied with the DFF).
- 2) It is recommended that areas of the welds where stress concentrations occur be marked as mandatory inspection areas for B, C and D categories as applicable.
- 3) Welds or parts of welds with no access for in-service inspection and repair should be assigned inspection category A.
- 4) Welds or parts of welds with no access for in-service inspection and repair should be assigned inspection category B.
- 5) Welds or parts of welds with no access for in-service inspection and repair should be assigned inspection category C.
- 6) High tensile stresses mean ULS tensile stresses in excess of 0.85 of design stress.
- 7) Moderate tensile stresses mean ULS tensile stresses between 0.6 and 0.85 of design stress.
- 8) Low tensile stresses mean ULS tensile stresses less than 0.6 of design stress.

Table 5-4 Determination of inspection category for details with high fatigue utilisation ¹⁾

Design Class	Direction of dominating principal stress	Inspection category ³⁾
DC1 and DC2	Welds with the direction of the dominating dynamic principal stress transverse to the weld (between 45° and 135°)	A ²⁾
	Welds with the direction of the dominating dynamic principal stress in the direction of the weld (between - 45° and 45°)	B ⁴⁾
DC3 and DC4	Welds with the direction of the dominating dynamic principal stress transverse to the weld (between 45° and 135°)	B ⁴⁾
	Welds with the direction of the dominating dynamic principal stress in the direction of the weld (between - 45° and 45°)	C ⁵⁾
DC5	Welds with the direction of the dominating dynamic principal stress transverse to the weld (between 45° and 135°)	D
	Welds with the direction of the dominating dynamic principal stress in the direction of the weld (between - 45° and 45°)	E

Notes:

- 1) High fatigue utilisation means connections with calculated fatigue life less than 3 times the required fatigue life (design fatigue life multiplied with the DFF).
- 2) Butt welds with high fatigue utilisation and SCF less than 1.3 need stricter NDT acceptance criteria. Such criteria need to be developed in each case.
- 3) For joints in inspection categories B, C or D, the hot spot regions (regions with highest stress range) at welds or areas of welds of special concern shall be addressed with individual notations as mandatory for selected NDT methods.
- 4) Welds or parts of welds with no access for in-service inspection and repair should be assigned inspection category A.
- 5) Welds or parts of welds with no access for in-service inspection and repair should be assigned inspection category B.

6 ULTIMATE LIMIT STATES

6.1 General

This clause gives provisions for checking of ULSs for typical structural elements used in offshore steel structures, where ordinary building codes lack relevant recommendations. Such elements are tubular members, tubular joints, conical transitions and some load situations for plates and stiffened plates. For other types of structural elements NS-EN 1993-1-1 for member design and NS-EN 1993-1-8 for other joints than tubular joints apply. See also sub clause 6.5.1 and 6.7.

The material factor γ_M is 1.15 for ULSs unless noted otherwise.

The material factors according to Table 6-1 shall be used if NS-EN 1993-1-1 and NS-EN 1993-1-8 are used for calculation of structural resistance:

Table 6-1 Material factors

Type of calculation	Material factor 1)	Value
Resistance of Class 1,2 or 3 cross-sections	γ_{M0}	1.15
Resistance of Class 4 cross-sections	γ_{M1}	1.15
Resistance of member to buckling	γ_{M1}	1.15
Resistance of net section at bolt holes	γ_{M2}	1.3
Resistance of fillet and partial penetration welds	γ_{M2}	1.3
Resistance of bolted connections	γ_{M2}	1.3

1) Symbols according to NS-EN 1993-1-1 and NS-EN 1993-1-8.

For resistance check where other material factors are used than given in Table 6-1 the recommended material factor in NS-EN 1993-1-1, NS-EN 1993-1-5 and NS-EN 1993-1-8 should be multiplied by an additional building code material factor $\gamma_{BC} = 1.05$

The ultimate strength of structural elements and systems should be evaluated by using a rational, justifiable engineering approach.

The structural analysis may be carried out as linear elastic, simplified rigid-plastic, or elastic-plastic analyses. Both first order or second order analyses may be applied. In all cases, the structural detailing with respect to strength and ductility requirements shall conform to the assumption made for the analysis.

When plastic or elastic-plastic analyses are used for structures exposed to cyclic loading (e.g. wave loads) checks shall be carried out to verify that the structure will shake down without excessive plastic deformations or fracture due to repeated yielding. A characteristic or design cyclic load history needs to be defined in such a way that the structural reliability in case of cyclic loading (e.g. storm loading) is not less than the structural reliability for ULSs for non-cyclic actions. It should be checked as a minimum that the structure will carry all loads throughout the entire storm comprising the ULS environmental condition.

In case of linear global beam analysis combined with the resistance formulations set down in this NORSOK standard, shakedown can be assumed without further checks.

6.2 Ductility

It is a fundamental requirement that all failure modes are sufficiently ductile such that the structural behaviour will be in accordance with the anticipated model used for determination of the responses. In general all design procedures, regardless of analysis method, will not capture the true structural behaviour. Ductile failure modes will allow the structure to redistribute forces in accordance with the presupposed static model. Brittle failure modes shall therefore be avoided or shall be verified to have excess resistance compared to ductile modes, and in this way protect the structure from brittle failure.

The following sources for brittle structural behaviour may need to be considered for a steel structure:

1. Unstable fracture caused by a combination of the following factors:
 - brittle material;

- a design resulting in high local stresses;
 - the possibilities for weld defects.
2. Structural details where ultimate resistance is reached with plastic deformations only in limited areas, making the global behaviour brittle, e.g. partial butt weld loaded transverse to the weld with failure in the weld.
 3. Shell buckling.
 4. Buckling where interaction between local and global buckling modes occur.

In general a steel structure will be of adequate ductility if the following is satisfied:

5. Material toughness requirements are met, and the design avoids a combination of high local stresses with possibilities of undetected weld defects.
6. Details are designed to develop a certain plastic deflection e.g. partial butt welds subjected to stresses transverse to the weld is designed with excess resistance compared with adjoining plates.
7. Member geometry is selected such that the resistance does not show a sudden drop in capacity when the member is subjected to deformation beyond maximum resistance. An unstiffened shell in cross-section class 4 is an example of a member that may show such an unfavourable resistance deformation relationship. For definition of cross-section class see NS-EN 1993-1-1.
8. Local and global buckling interaction effects are avoided.

6.3 Tubular members

6.3.1 General

The structural strength and stability requirements for steel tubular members are specified in this section.

The requirements given in this section apply to un stiffened and ring stiffened tubulars having a thickness $t \geq 6$ mm, $D/t < 120$ and material meeting the general requirements in Clause 5. In cases where hydrostatic pressure are present, the structural analysis may proceed on the basis that stresses due to the capped-end forces arising from hydrostatic pressure are either included in or excluded from the analysis. This aspect is discussed in Clause 12.

In the following subclauses, y and z are used to define the in-plane and out-of-plane axes of a tubular member, respectively.

The requirements assume the tubular is constructed in accordance with the fabrication tolerances given in NORSOK M-101.

The requirements are formulated for an isolated beam column. This formulation may also be used to check the resistance of frames and trusses, provided that each member is checked for the member forces and moments combined with a representative effective length. The effective length may in lieu of special analyses be determined according to the requirements given in this chapter. Alternatively the ULSs for frames or trusses may be determined on basis of non-linear analyses taking into account second order effects. The use of these analyses requires that the assumptions made are fulfilled and justified.

Tubular members subjected solely to axial tension, axial compression, bending, shear, or hydrostatic pressure should be designed to satisfy the strength and stability requirements specified in 6.3.2 to 6.3.6. Tubular members subjected to combined loads without hydrostatic pressure should be designed to satisfy the strength and stability requirements specified in 6.3.8. Tubular members subjected to combined loads with hydrostatic pressure should be designed to satisfy the strength and stability requirements specified in 6.3.9.

The equations in this section are not using an unique sign convention. Definitions are given in each paragraph.

6.3.2 Axial tension

Tubular members subjected to axial tensile loads should be designed to satisfy the following condition:

$$N_{Sd} \leq N_{t,Rd} = \frac{A f_y}{\gamma_M} \quad (6.1)$$

where

N_{Sd}	=	design axial force (tension positive)
f_y	=	characteristic yield strength
A	=	cross sectional area
γ_M	=	1.15.

6.3.3 Axial compression

Tubular members subjected to axial compressive loads should be designed to satisfy the following condition:

$$N_{Sd} \leq N_{c,Rd} = \frac{Af_c}{\gamma_M} \quad (6.2)$$

where

N_{Sd}	=	design axial force (compression positive)
f_c	=	characteristic axial compressive strength
γ_M	=	see 6.3.7

In the absence of hydrostatic pressure the characteristic axial compressive strength for tubular members shall be the smaller of the in-plane or out-of-plane buckling strength determined from the following equations:

$$f_c = [1.0 - 0.28\bar{\lambda}^2]f_{cl} \quad \text{for } \bar{\lambda} \leq 1.34 \quad (6.3)$$

$$f_c = \frac{0.9}{\bar{\lambda}^2}f_{cl} \quad \text{for } \bar{\lambda} > 1.34 \quad (6.4)$$

$$\bar{\lambda} = \sqrt{\frac{f_{cl}}{f_E}} = \frac{kl}{\pi i} \sqrt{\frac{f_{cl}}{E}} \quad (6.5)$$

where

f_{cl}	=	characteristic local buckling strength
$\bar{\lambda}$	=	column slenderness parameter
f_E	=	smaller Euler buckling strength in y or z direction
E	=	Young's modulus of elasticity, $2.1 \cdot 10^5$ MPa
k	=	effective length factor, see 6.3.8.2
l	=	longer unbraced length in y or z direction
i	=	radius of gyration

The characteristic local buckling strength should be determined from:

$$f_{cl} = f_y \quad \text{for } \frac{f_y}{f_{cle}} \leq 0.170 \quad (6.6)$$

$$f_{cl} = \left(1.047 - 0.274 \frac{f_y}{f_{cle}}\right)f_y \quad \text{for } 0.170 < \frac{f_y}{f_{cle}} \leq 1.911 \quad (6.7)$$

$$f_{cl} = f_{cle} \quad \text{for } \frac{f_y}{f_{cle}} > 1.911 \quad (6.8)$$

and

$$f_{cle} = 2C_e E \frac{t}{D}$$

where

f_{cle}	=	characteristic elastic local buckling strength
C_e	=	critical elastic buckling coefficient = 0.3
D	=	outside diameter
t	=	wall thickness

For $\frac{f_y}{f_{cle}} > 0.170$ the tubular is a class 4 cross section and may behave as a shell. Shell structures may have a brittle structure failure mode. Reference is made to 6.2. For class 4 cross sections increased γ_M values shall be used according to Equation (6.22).

6.3.4 Bending

Tubular members subjected to bending loads should be designed to satisfy the following condition:

$$M_{Sd} \leq M_{Rd} = \frac{f_m W}{\gamma_M} \quad (6.9)$$

where

M_{Sd}	=	design bending moment
f_m	=	characteristic bending strength
W	=	elastic section modulus
γ_M	=	see section 6.3.7

The characteristic bending strength for tubular members should be determined from:

$$f_m = \frac{Z}{W} f_y \quad \text{for } \frac{f_y D}{Et} \leq 0.0517 \quad (6.10)$$

$$f_m = \left(1.13 - 2.58 \left(\frac{f_y D}{Et} \right) \right) \left(\frac{Z}{W} \right) f_y \quad 0.0517 < \frac{f_y D}{Et} \leq 0.1034 \quad (6.11)$$

$$f_m = \left(0.94 - 0.76 \left(\frac{f_y D}{Et} \right) \right) \left(\frac{Z}{W} \right) f_y \quad 0.1034 < \frac{f_y D}{Et} \leq 120 \frac{f_y}{E} \quad (6.12)$$

where

W	=	elastic section modulus
	=	$\frac{\pi [D^4 - (D - 2t)^4]}{32 D}$
Z	=	plastic section modulus
	=	$\frac{1}{6} [D^3 - (D - 2t)^3]$

For $\frac{f_y}{f_{cle}} > 0.170$ the tubular is a class 4 cross section and may behave as a shell. Shell structures may have a brittle structure failure mode. Reference is made to 6.2. For class 4 cross sections increased γ_M values shall be used according to Equation (6.22).

6.3.5 Shear

Tubular members subjected to beam shear forces should be designed to satisfy the following condition:

$$V_{Sd} \leq V_{Rd} = \frac{A f_y}{2\sqrt{3} \gamma_M} \quad (6.13)$$

where

$$\begin{aligned} V_{Sd} &= \text{design shear force} \\ f_y &= \text{yield strength} \\ A &= \text{cross sectional area} \\ \gamma_M &= 1.15 \end{aligned}$$

Tubular members subjected to shear from torsional moment should be designed to satisfy the following condition:

$$M_{T,Sd} \leq M_{T,Rd} = \frac{2I_p f_y}{D\sqrt{3} \gamma_M} \quad (6.14)$$

where

$$\begin{aligned} M_{T,Sd} &= \text{design torsional moment} \\ I_p &= \text{polar moment of inertia} = \frac{\pi}{32} [D^4 - (D - 2t)^4] \end{aligned}$$

6.3.6 Hydrostatic pressure

6.3.6.1 Hoop buckling

Tubular members subjected to external pressure should be designed to satisfy the following condition:

$$\sigma_{p,Sd} \leq f_{h,Rd} = \frac{f_h}{\gamma_M} \quad (6.15)$$

$$\sigma_{p,Sd} = \frac{p_{Sd} D}{2t} \quad (6.16)$$

where

$$\begin{aligned} f_h &= \text{characteristic hoop buckling strength} \\ \sigma_{p,Sd} &= \text{design hoop stress due to hydrostatic pressure (compression positive)} \\ p_{Sd} &= \text{design hydrostatic pressure} \\ \gamma_M &= \text{see 6.3.7} \end{aligned}$$

If out-of-roundness tolerances do not meet the requirements given in NORSOK M-101, guidance on calculating reduced strength is given in Clause 12.

$$f_h = f_y, \quad \text{for } f_{he} > 2.44f_y \quad (6.17)$$

$$f_h = 0.7f_y \left[\frac{f_{he}}{f_y} \right]^{0.4} \quad \text{for } 2.44f_y \geq f_{he} > 0.55f_y \quad (6.18)$$

$$f_h = f_{he}, \quad \text{for } f_{he} \leq 0.55f_y \quad (6.19)$$

The elastic hoop buckling strength, f_{he} , is determined from the following equation:

$$f_{he} = 2C_h E \frac{t}{D} \quad (6.20)$$

where

$$\begin{aligned} C_h &= 0.44 t/D \text{ for } \mu \geq 1.6D/t \\ &= 0.44 t/D + 0.21 (D/t)^3 / \mu^4 \quad \text{for } 0.825D/t \leq \mu < 1.6D/t \\ &= 0.737 / (\mu - 0.579) \quad \text{for } 1.5 \leq \mu < 0.825D/t \end{aligned}$$

$$= 0.80 \quad \text{for } \mu < 1.5$$

and where the geometric parameter, μ , is defined as:

$$\mu = \frac{L}{D} \sqrt{\frac{2D}{t}} \quad \text{and}$$

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

6.3.6.2 Ring stiffener design

The circumferential stiffening ring size may be selected on the following approximate basis:

$$I_c = f_{he} \frac{t L_r D^2}{8E} \quad (6.21)$$

where

I_c = required moment of inertia for ring composite section
 L_r = ring spacing
 D = diameter (see Note 2 for external rings)

Notes:

1. Equation (6.21) assumes that the yield strength of the stiffening ring is equal to or greater than that of the tubular.
2. For external rings, D in Equation (6.21) should be taken to the centroid of the composite ring.
3. An effective width of shell equal to $1.1\sqrt{D \cdot t}$ may be assumed as the flange for the composite ring section.
4. Where out-of-roundness in excess of tolerances given in NORSOK M-101 is permitted, larger stiffeners may be required. The bending due to out-of-roundness should be specially investigated.

Local buckling of ring stiffeners with flanges may be excluded as a possible failure mode provided that the following requirements are fulfilled:

$$\frac{h}{t_w} \leq 1.1 \sqrt{\frac{E}{f_y}}$$

and

$$\frac{b}{t_f} \leq 0.3 \sqrt{\frac{E}{f_y}}$$

where

h = web height
 t_w = web thickness
 b = half the width of flange of T-stiffeners
 t_f = thickness of flange

Local buckling of ring stiffeners without flanges may be excluded as a possible failure mode provided that:

$$\frac{h}{t_w} \leq 0.4 \sqrt{\frac{E}{f_y}}$$

Torsional buckling of ring stiffeners with flanges may be excluded as a possible failure mode provided that:

$$b \geq \frac{3.5 h}{\sqrt{10 + \frac{h E}{r f_y}}}$$

Check for torsional buckling of the stiffener shall be made according to DNV RP-C202 /13/.

6.3.7 Material factor

The material factor, γ_M , is given as:

$$\begin{aligned}
\gamma_M &= 1.15 & \text{for } \bar{\lambda}_s < 0.5 \\
\gamma_M &= 0.85 + 0.60\bar{\lambda}_s & \text{for } 0.5 \leq \bar{\lambda}_s \leq 1.0 \\
\gamma_M &= 1.45 & \text{for } \bar{\lambda}_s > 1.0
\end{aligned} \tag{6.22}$$

where

$$\bar{\lambda}_s = \frac{|\sigma_{c,Sd}|}{f_{cl}} \cdot \lambda_c + \left(\frac{\sigma_{p,Sd}}{f_h} \right)^2 \cdot \lambda_h \tag{6.23}$$

where f_{cl} is calculated from Equation (6.6) or Equation (6.7) whichever is appropriate and f_h from Equation (6.17), Equation (6.18), or Equation (6.19) whichever is appropriate.

$$\lambda_c = \sqrt{\frac{f_y}{f_{cle}}}, \text{ and } \lambda_h = \sqrt{\frac{f_y}{f_{he}}} \tag{6.24}$$

f_{cle} and f_{he} is obtained from Equation (6.8), and Equation (6.20) respectively.

$\sigma_{p,Sd}$ is obtained from Equation (6.16) and

$$\sigma_{c,Sd} = \frac{N_{Sd}}{A} + \frac{\sqrt{M_{y,Sd}^2 + M_{z,Sd}^2}}{W} \tag{6.25}$$

N_{Sd} is negative if in tension.

6.3.8 Tubular members subjected to combined loads without hydrostatic pressure

6.3.8.1 Axial tension and bending

Tubular members subjected to combined axial tension and bending loads should be designed to satisfy the following condition at all cross sections along their length:

$$\left(\frac{N_{Sd}}{N_{t,Rd}} \right)^{1.75} + \frac{\sqrt{M_{y,Sd}^2 + M_{z,Sd}^2}}{M_{Rd}} \leq 1.0 \tag{6.26}$$

where

$$\begin{aligned}
M_{y,Sd} &= \text{design bending moment about member y-axis (in-plane)} \\
M_{z,Sd} &= \text{design bending moment about member z-axis (out-of-plane)} \\
N_{Sd} &= \text{design axial tensile force}
\end{aligned}$$

If shear or torsion is of importance, the bending capacity M_{Rd} needs to be substituted with $M_{Red,Rd}$ calculated according to subclause 6.3.8.3 or 6.3.8.4.

6.3.8.2 Axial compression and bending

Tubular members subjected to combined axial compression and bending should be designed to satisfy the following condition accounting for possible variations in cross-section, axial load and bending moment according to appropriate engineering principles:

$$\frac{N_{Sd}}{N_{c,Rd}} + \frac{1}{M_{Rd}} \left\{ \left[\frac{C_{my} M_{y,Sd}}{1 - \frac{N_{Sd}}{N_{Ey}}} \right]^2 + \left[\frac{C_{mz} M_{z,Sd}}{1 - \frac{N_{Sd}}{N_{Ez}}} \right]^2 \right\}^{0.5} \leq 1.0 \tag{6.27}$$

and at all cross sections along their length:

$$\frac{N_{Sd}}{N_{cl,Rd}} + \frac{\sqrt{M_{y,Sd}^2 + M_{z,Sd}^2}}{M_{Rd}} \leq 1.0 \quad (6.28)$$

where

$$\begin{aligned} N_{Sd} &= \text{design axial compression force} \\ C_{my}, C_{mz} &= \text{reduction factors corresponding to the member y and z axes, respectively} \\ N_{Ey}, N_{Ez} &= \text{Euler buckling strengths corresponding to the member y and z axes, respectively} \\ N_{cl,Rd} &= \frac{f_{cl} \cdot A}{\gamma_M} \quad \text{design axial local buckling resistance} \end{aligned}$$

$$N_{Ey} = \frac{\pi^2 EA}{\left[\frac{kl}{i} \right]_y^2} \quad (6.29)$$

$$N_{Ez} = \frac{\pi^2 EA}{\left[\frac{kl}{i} \right]_z^2} \quad (6.30)$$

k in Equation (6.29) and Equation (6.30) relate to buckling in the y and z directions, respectively.

These factors can be determined using a rational analysis that includes joint flexibility and side-sway. In lieu of such a rational analysis, values of effective length factors, k, and moment reduction factors, C_m , may be taken from Table 6-2. All lengths are measured centreline to centreline.

Table 6-2 Effective length and moment reduction factors for member strength checking

Structural element	k	$C_m^{(1)}$
<i>Superstructure legs</i>		
- Braced	1.0	(a)
- Portal (unbraced)	$k^{(2)}$	(a)
<i>Jacket legs and piling</i>		
- Grouted composite section	1.0	(c)
- UngROUTED jacket legs	1.0	(c)
- UngROUTED piling between shim points	1.0	(b)
<i>Jacket braces</i>		
- Primary diagonals and horizontals	0.7	(b) or (c)
- K-braces ⁽³⁾	0.7	(c)
- Longer segment length of X-braces ⁽³⁾	0.8	(c)
<i>Secondary horizontals</i>	0.7	(c)
Notes:		
1. C_m values for the cases defined in Table 6-2 are as follows: (a) 0.85 (b) for members with no transverse loading, $C_m = 0.6 - 0.4 M_{1,Sd}/M_{2,Sd}$, where $M_{1,Sd}/M_{2,Sd}$ is the ratio of smaller to larger moments at the ends of that portion of the member unbraced in the plane of bending under consideration. $M_{1,Sd}/M_{2,Sd}$ is positive when the number is bent in reverse curvature, negative when bent in single curvature. (c) for members with transverse loading, $C_m = 1.0 - 0.4 N_{Sd}/N_E$, or 0.85, whichever is less, and $N_E = N_{Ey}$ or N_{Ez} as appropriate.		
2. Use effective length alignment chart in Clause 12.		
3. At least one pair of members framing into the a K- or X-joint shall be in tension if the joint is not braced out-of-plane. For X-braces, when all members are in compression, the k-factor should be determined using the procedures given in Clause 12.		
4. The effective length and C_m factors given in Table 6-2 do not apply to cantilever members and the member ends are assumed to be rotationally restrained in both planes of bending.		

6.3.8.3 Interaction shear and bending moment

Tubular members subjected to beam shear force and bending moment should be designed to satisfy the following condition provided that the direction of the shear force and the moment vectors are orthogonal within $\pm 20^\circ$:

$$\frac{M_{Sd}}{M_{Rd}} \leq \sqrt{1.4 - \frac{V_{Sd}}{V_{Rd}}} \quad \text{for } \frac{V_{Sd}}{V_{Rd}} \geq 0.4 \quad (6.31)$$

$$\frac{M_{Sd}}{M_{Rd}} \leq 1.0 \quad \text{for } \frac{V_{Sd}}{V_{Rd}} < 0.4 \quad (6.32)$$

6.3.8.4 Interaction shear, bending moment and torsional moment

Tubular members subjected to beam shear force, bending moment and torsion should be designed to satisfy the following condition provided that the direction of the shear force and the moment vectors are orthogonal within $\pm 20^\circ$:

$$\frac{M_{Sd}}{M_{Red,Rd}} \leq \sqrt{1.4 - \frac{V_{Sd}}{V_{Rd}}} \quad \text{for } \frac{V_{Sd}}{V_{Rd}} \geq 0.4 \quad (6.33)$$

$$\frac{M_{Sd}}{M_{Red,Rd}} \leq 1.0 \quad \text{for } \frac{V_{Sd}}{V_{Rd}} < 0.4$$

where

$$\begin{aligned} M_{Red,Rd} &= \frac{W f_{m,Red}}{\gamma_M} \\ f_{m,Red} &= f_m \sqrt{1 - 3 \left(\frac{\tau_{T,Sd}}{f_d} \right)^2} \\ \tau_{T,Sd} &= \frac{M_{T,Sd}}{2\pi R^2 t} \\ f_d &= \frac{f_y}{\gamma_M} \\ R &= \text{radius of tubular member} \\ \gamma_M &= \text{see 6.3.7} \end{aligned}$$

6.3.9 Tubular members subjected to combined loads with hydrostatic pressure

The design provisions in this section are divided into two categories. In Method A it is assumed that the capped-end compressive forces due to the external hydrostatic pressure are not included in the structural analysis. Alternatively, the design provisions in Method B assume that such forces are included in the analysis as external nodal forces. Dependent upon the method used in the analysis, the interaction equations in either Method A or Method B should be satisfied at all cross sections along their length.

It should be noted that the equations in this section are not applicable unless Equation (6.15) is first satisfied.

For guidance on significance of hydrostatic pressure, see Clause 12.

6.3.9.1 Axial tension, bending, and hydrostatic pressure

Tubular members subjected to combined axial tension, bending, and hydrostatic pressure should be designed to satisfy the following equations in either Method A or Method B at all cross sections along their length.

Method A ($\sigma_{a,Sd}$ is in tension)

In this method, the calculated value of member axial stress, $\sigma_{a,Sd}$, should not include the effect of the hydrostatic capped-end axial stress. The capped-end axial compression due to external hydrostatic pressure, $\sigma_{q,Sd}$, can be taken as $0.5\sigma_{p,Sd}$. This implies that, the tubular member takes the entire capped-end force arising from external hydrostatic pressure. In reality, the stresses in the member due to this force depend on the restraint provided by the rest of the structure on the member. The stress computed from a more rigorous analysis may be substituted for $0.5\sigma_{p,Sd}$.

(a) For $\sigma_{a,Sd} \geq \sigma_{q,Sd}$ (net axial tension condition)

$$\frac{\sigma_{a,Sd} - \sigma_{q,Sd}}{f_{th,Rd}} + \frac{\sqrt{\sigma_{my,Sd}^2 + \sigma_{mz,Sd}^2}}{f_{mh,Rd}} \leq 1.0 \quad (6.34)$$

where

- $\sigma_{a,Sd}$ = design axial stress that excludes the effect of capped-end axial compression arising from external hydrostatic pressure (tension positive)
- $\sigma_{q,Sd}$ = capped-end design axial compression stress due to external hydrostatic pressure (= $0.5\sigma_{p,Sd}$) (compression positive)
- $\sigma_{my,Sd}$ = design in plane bending stress
- $\sigma_{mz,Sd}$ = design out of plane bending stress
- $f_{th,Rd}$ = design axial tensile resistance in the presence of external hydrostatic pressure which is given by Equation (6.35):

$$f_{th,Rd} = \frac{f_y}{\gamma_M} [\sqrt{1 + 0.09B^2 - B^{2\eta}} - 0.3B] \quad (6.35)$$

- $f_{mh,Rd}$ = design bending resistance in the presence of external hydrostatic pressure which is given by Equation (6.36):

$$f_{mh,Rd} = \frac{f_m}{\gamma_M} [\sqrt{1 + 0.09B^2 - B^{2\eta}} - 0.3B] \quad (6.36)$$

$$\gamma_M = \text{see 6.3.7}$$

and Equation (6.37):

$$B = \frac{\sigma_{p,Sd}}{f_{h,Rd}}, \quad B \leq 1.0 \quad (6.37)$$

$$\eta = 5 - 4 \frac{f_h}{f_y} \quad (6.38)$$

(b) For $\sigma_{a,Sd} < \sigma_{q,Sd}$ (net axial compression condition)

$$\frac{|\sigma_{a,Sd} - \sigma_{q,Sd}|}{f_{cl,Rd}} + \frac{\sqrt{\sigma_{my,Sd}^2 + \sigma_{mz,Sd}^2}}{f_{mh,Rd}} \leq 1.0 \quad (6.39)$$

$$f_{cl,Rd} = \frac{f_{cl}}{\gamma_M} \quad (6.40)$$

where f_{cl} is found from Equation (6.6) and Equation (6.7).

When $\sigma_{c,Sd} > 0.5 \frac{f_{he}}{\gamma_M}$ and $f_{cle} > 0.5 f_{he}$, Equation (6.41) should also be satisfied:

$$\frac{\sigma_{c,Sd} - 0.5 \frac{f_{he}}{\gamma_M}}{\frac{f_{cle} - 0.5 \frac{f_{he}}{\gamma_M}}{\gamma_M}} + \left[\frac{\sigma_{p,Sd}}{\frac{f_{he}}{\gamma_M}} \right]^2 \leq 1.0 \quad (6.41)$$

in which $\sigma_{c,Sd} = \sigma_{m,Sd} + \sigma_{q,Sd} - \sigma_{a,Sd}$; $\sigma_{c,Sd}$ should reflect the maximum combined compressive stress.

$$\sigma_{m,Sd} = \frac{\sqrt{M_{z,Sd}^2 + M_{y,Sd}^2}}{W}$$

$\gamma_M =$ see 6.3.7

Method B ($\sigma_{ac,Sd}$ is in tension)

In this method, the calculated member axial stress, $\sigma_{ac,Sd}$, includes the effect of the hydrostatic capped-end axial stress. Only Equation (6.42) needs to be satisfied:

$$\frac{\sigma_{ac,Sd}}{f_{th,Rd}} + \frac{\sqrt{\sigma_{my,Sd}^2 + \sigma_{mz,Sd}^2}}{f_{mh,Rd}} \leq 1.0 \quad (6.42)$$

where

$\sigma_{ac,Sd} =$ design axial stress that includes the effect of the capped-end compression arising from external hydrostatic pressure (tension positive)

6.3.9.2 Axial compression, bending, and hydrostatic pressure

Tubular members subjected to combined compression, bending, and hydrostatic pressure should be proportioned to satisfy the following requirements at all cross sections along their length.

Method A ($\sigma_{a,Sd}$ is in compression)

$$\frac{\sigma_{a,Sd}}{f_{ch,Rd}} + \frac{1}{f_{mh,Rd}} \left[\left(\frac{C_{my} \sigma_{my,Sd}}{1 - \frac{\sigma_{a,Sd}}{f_{Ey}}} \right)^2 + \left(\frac{C_{mz} \sigma_{mz,Sd}}{1 - \frac{\sigma_{a,Sd}}{f_{Ez}}} \right)^2 \right]^{0.5} \leq 1.0 \quad (6.43)$$

$$\frac{\sigma_{a,Sd} + \sigma_{q,Sd}}{f_{cl,Rd}} + \frac{\sqrt{\sigma_{my,Sd}^2 + \sigma_{mz,Sd}^2}}{f_{mh,Rd}} \leq 1.0 \quad (6.44)$$

where

$\sigma_{a,Sd} =$ design axial stress that excludes the effect of capped-end axial compression arising from external hydrostatic pressure (compression positive)

$$f_{Ey} = \frac{\pi^2 E}{\left[\frac{kl}{i} \right]_y^2} \quad (6.45)$$

$$f_{Ez} = \frac{\pi^2 E}{\left[\frac{kl}{i} \right]_z^2} \quad (6.46)$$

$f_{ch,Rd}$ = design axial compression strength in the presence of external hydrostatic pressure which is given by the following equations:

$$f_{ch,Rd} = \frac{1}{2} \frac{f_{cl}}{\gamma_M} \left[\xi - \frac{2\sigma_{q,Sd}}{f_{cl}} + \sqrt{\xi^2 + 1.12 \bar{\lambda}^2 \frac{\sigma_{q,Sd}}{f_{cl}}} \right] \quad \text{for } \bar{\lambda} < 1.34 \sqrt{\left[\left(1 - \frac{2\sigma_{q,Sd}}{f_{cl}} \right) \right]^{-1}} \quad (6.47)$$

and

$$f_{ch,Rd} = \frac{0.9 f_{cl}}{\bar{\lambda}^2 \gamma_M}, \quad \text{for } \bar{\lambda} \geq 1.34 \sqrt{\left[\left(1 - \frac{2\sigma_{q,Sd}}{f_{cl}} \right) \right]^{-1}} \quad (6.48)$$

where

$$\xi = 1 - 0.28 \bar{\lambda}^2 \quad (6.49)$$

When $\sigma_{c,Sd} > 0.5 \frac{f_{he}}{\gamma_M}$ and $f_{cle} > 0.5 f_{he}$, Equation (6.41), in which $\sigma_{c,Sd} = \sigma_{m,Sd} + \sigma_{q,Sd} + \sigma_{a,Sd}$, should also be satisfied.

γ_M = see 6.3.7

Method B ($\sigma_{ac,Sd}$ is in compression)

(a) for $\sigma_{ac,Sd} > \sigma_{q,Sd}$

$$\frac{\sigma_{ac,Sd} - \sigma_{q,Sd}}{f_{ch,Rd}} + \frac{1}{f_{mh,Rd}} \left[\left(\frac{C_{my} \sigma_{my,Sd}}{1 - \frac{\sigma_{ac,Sd} - \sigma_{q,Sd}}{f_{Ey}}} \right)^2 + \left(\frac{C_{mz} \sigma_{mz,Sd}}{1 - \frac{\sigma_{ac,Sd} - \sigma_{q,Sd}}{f_{Ez}}} \right)^2 \right]^{0.5} \leq 1.0 \quad (6.50)$$

$$\frac{\sigma_{ac,Sd}}{f_{cl,Rd}} + \frac{\sqrt{\sigma_{my,Sd}^2 + \sigma_{mz,Sd}^2}}{f_{mh,Rd}} \leq 1.0 \quad (6.51)$$

$\sigma_{ac,Sd}$ = design axial stress that includes the effect of capped-end axial compression arising from external hydrostatic pressure (compression positive)

When $\sigma_{c,Sd} > 0.5 \frac{f_{he}}{\gamma_M}$ and $\frac{f_{cle}}{\gamma_M} > 0.5 \frac{f_{he}}{\gamma_M}$, Equation (6.41), in which $\sigma_{c,Sd} = \sigma_{m,Sd} + \sigma_{ac,Sd}$, should also be satisfied.

(b) for $\sigma_{ac,Sd} \leq \sigma_{q,Sd}$

For $\sigma_{ac,Sd} \leq \sigma_{q,Sd}$, Equation (6.51) should be satisfied.

When $\sigma_{c,Sd} > 0.5 \frac{f_{he}}{\gamma_M}$ and $\frac{f_{cle}}{\gamma_M} > 0.5 \frac{f_{he}}{\gamma_M}$, Equation (6.41), in which $\sigma_{c,Sd} = \sigma_{m,Sd} + \sigma_{ac,Sd}$, should also be satisfied.

$\gamma_M =$ see 6.3.7

6.4 Tubular joints

6.4.1 General

The following provisions apply to the design of tubular joints formed by the connection of two or more members. Terminology for simple joints is defined in Figure 6-1. Figure 6-1 also gives some design requirements with respect to joint geometry. The gap for simple K-joints should be larger than 50 mm and less than D. Minimum distances for chord cans and brace stubs should not include thickness tapers.

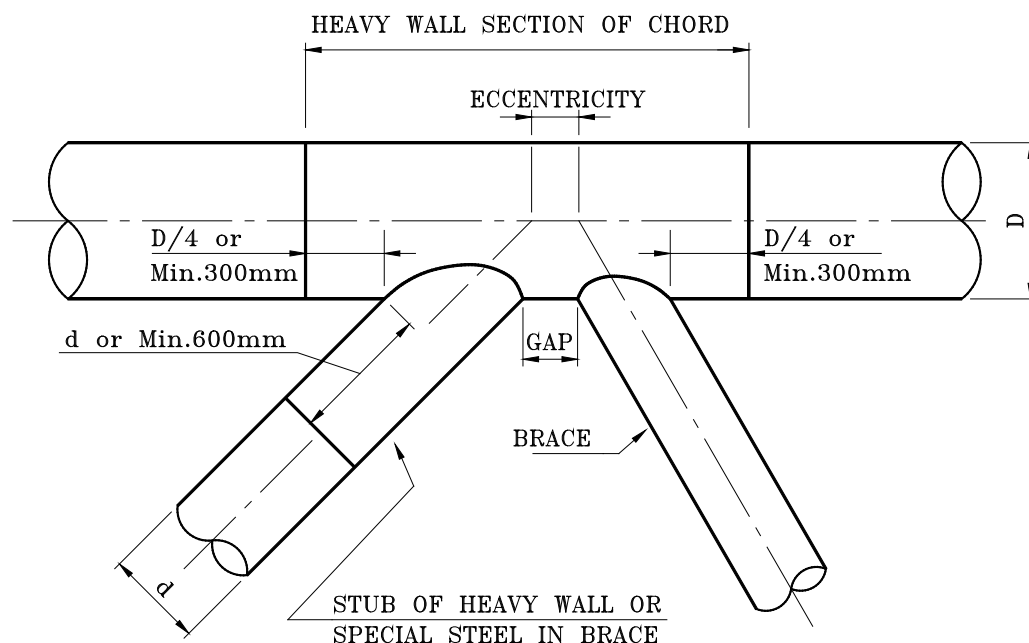


Figure 6-1 Detail of simple joint

Reductions in secondary (deflection induced) bending moments or inelastic relaxation through use of joint elastic stiffness may be considered. In certain instances, hydrostatic pressure effects may be significant.

6.4.2 Joint classification

Joint classification is the process whereby the axial force in a given brace is subdivided into K, X and Y components of actions, corresponding to the three joint types for which resistance equations exist. Such subdivision normally considers all of the members in one plane at a joint. For purposes of this provision, brace planes within $\pm 15^\circ$ of each other may be considered as being in a common plane. Each brace in the plane can have a unique classification that could vary with action condition. The classification can be a

mixture between the above three joint types. Once the breakdown into axial components is established, the resistance of the joint can be estimated using the procedures in 6.4.3.

Figure 6-2 provides some simple examples of joint classification. For a brace to be considered as K-joint classification, the axial force in the brace should be balanced to within 10 % by forces in other braces in the same plane and on the same side of the joint. For Y-joint classification, the axial force in the brace is reacted as beam shear in the chord. For X-joint classification, the axial force in the brace is carried through the chord to braces on the opposite side.

Additional explanation of joint-classification is found in Clause 12.

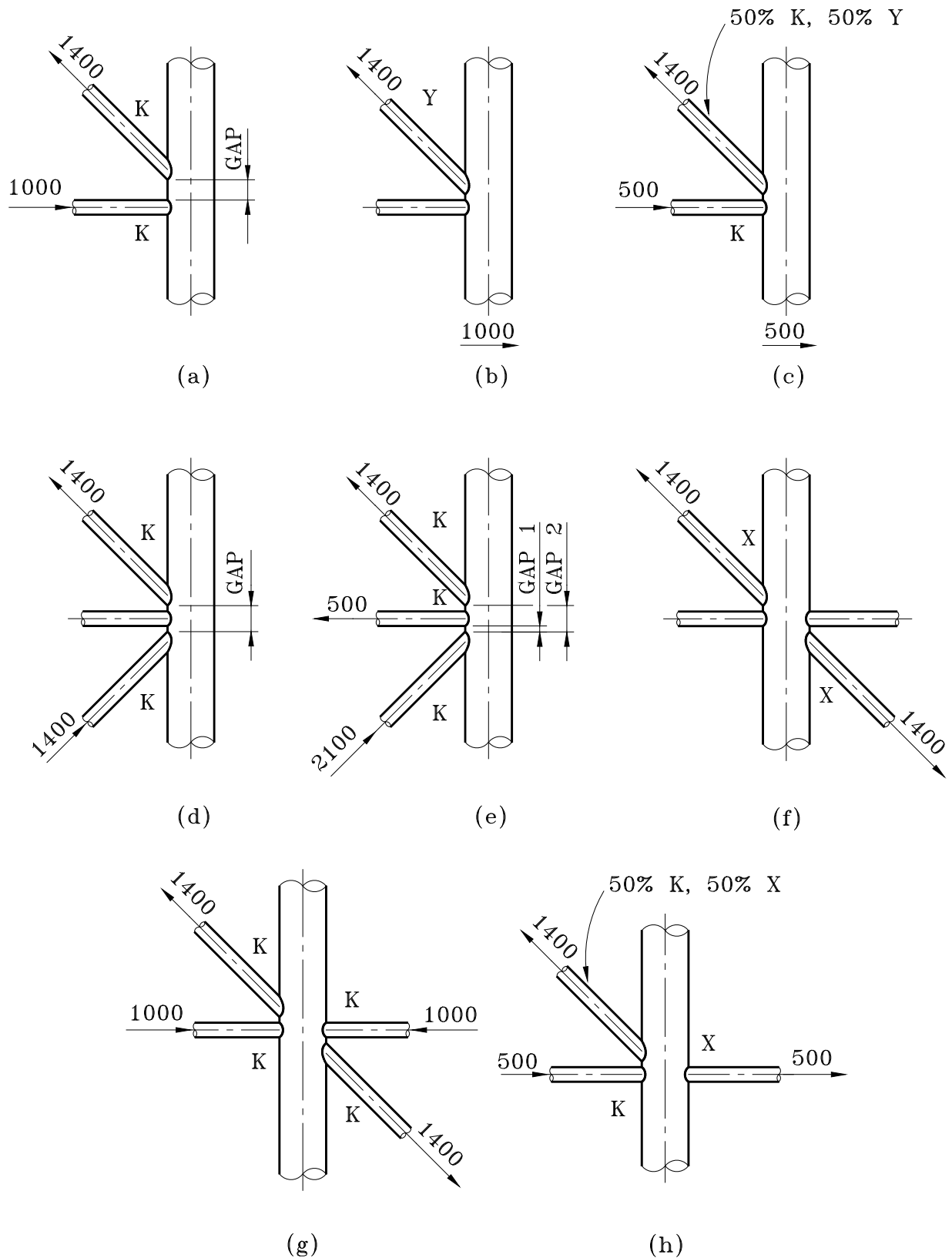


Figure 6-2 Classification of simple joints

6.4.3 Strength of simple joints

6.4.3.1 General

The validity range for application of the equations defined in 6.4.3 is as follows:

$$0.2 \leq \beta \leq 1.0$$

$$10 \leq \gamma \leq 50$$

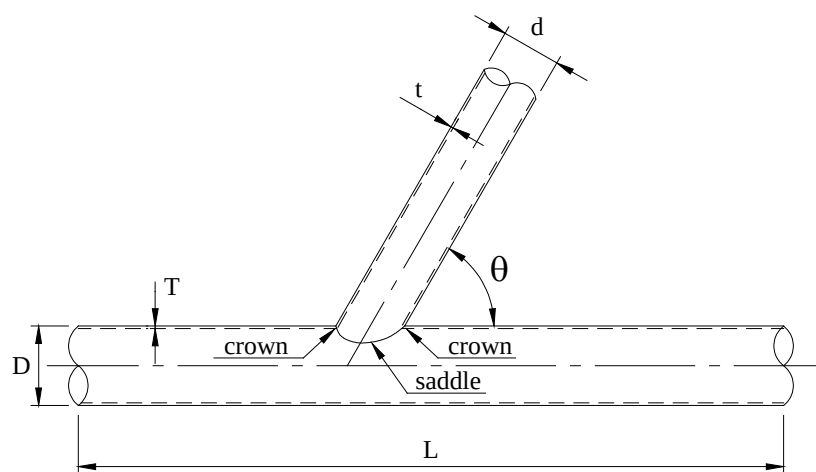
$$30^\circ \leq \theta \leq 90^\circ$$

$$\frac{g}{D} \geq -0.6 \text{ (for K joints)}$$

The equations can be used for joints with geometries which lie outside the validity ranges, by taking the usable strength as the lesser of the capacities calculated on the basis of:

- actual geometric parameters,
- imposed limiting parameters for the validity range, where these limits are infringed.

The above geometry parameters are defined in Figure 6-3 to Figure 6-6.

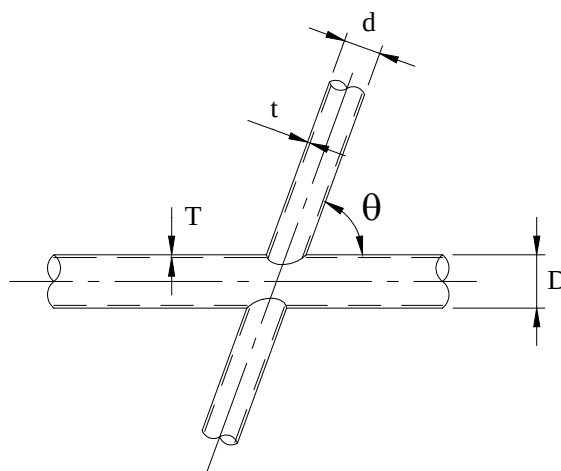


$$\beta = \frac{d}{D}$$

$$\gamma = \frac{D}{2T}$$

$$\tau = \frac{t}{T}$$

Figure 6-3 Definition of geometrical parameters for T- or Y-joints

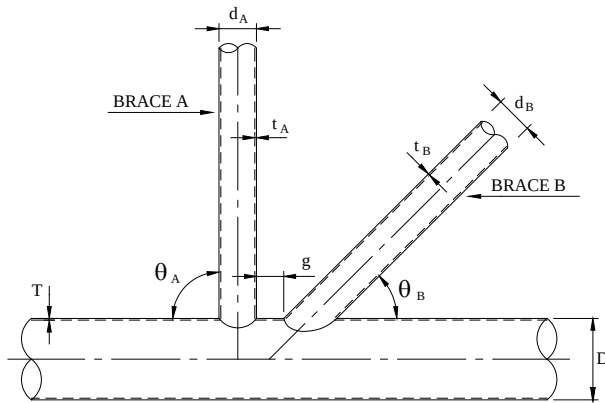


$$\beta = \frac{d}{D}$$

$$\gamma = \frac{D}{2T}$$

$$\tau = \frac{t}{T}$$

Figure 6-4 Definition of geometrical parameters for X-joints



$$\beta_A = \frac{d_A}{D}$$

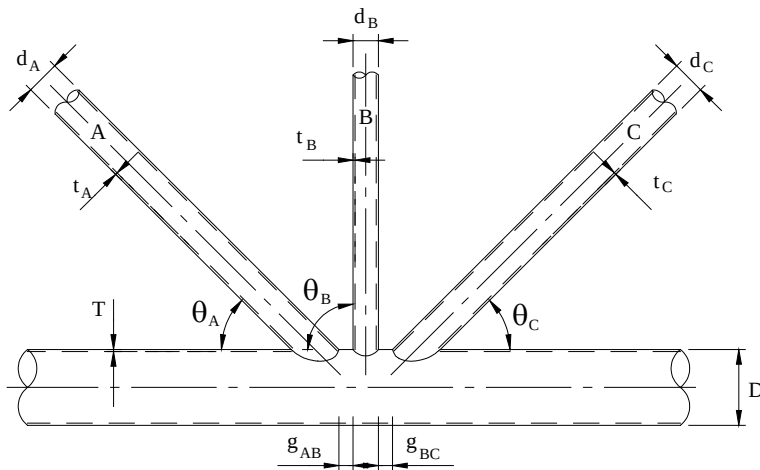
$$\beta_B = \frac{d_B}{D}$$

$$\tau_A = \frac{t_A}{T}$$

$$\tau_B = \frac{t_B}{T}$$

$$\gamma = \frac{D}{2T}$$

Figure 6-5 Definition of geometrical parameters for K-joints



$$\beta_A = \frac{d_A}{D}$$

$$\beta_B = \frac{d_B}{D}$$

$$\beta_C = \frac{d_C}{D}$$

$$\tau_A = \frac{t_A}{T}$$

$$\tau_B = \frac{t_B}{T}$$

$$\tau_C = \frac{t_C}{T}$$

$$\gamma = \frac{D}{2T}$$

Figure 6-6 Definition of geometrical parameters for KT-joints

6.4.3.2 Basic resistance

Tubular joints without overlap of principal braces and having no gussets, diaphragms, grout, or stiffeners should be designed using the following guidelines.

The characteristic resistances for simple tubular joints are defined as follows:

$$N_{Rd} = \frac{f_y T^2}{\gamma_M \sin \theta} Q_u Q_f \quad (6.52)$$

$$M_{Rd} = \frac{f_y T^2 d}{\gamma_M \sin \theta} Q_u Q_f \quad (6.53)$$

where

N_{Rd}	=	the joint design axial resistance
M_{Rd}	=	the joint design bending moment resistance
f_y	=	the yield strength of the chord member at the joint
γ_M	=	1.15

For joints with joint cans, N_{Rd} shall not exceed the resistance limits defined in 6.4.3.5

For braces with axial forces with a classification that is a mixture of K, Y and X joints, a weighted average of N_{Rd} based on the portion of each in the total action is used to calculate the resistance.

6.4.3.3 Strength factor Q_u

Q_u varies with the joint and action type, as given in Table 6-3.

Table 6-3 Values for Q_u

Joint Classification	Brace action			
	Axial Tension	Axial Compression	In-plane Bending	Out-of-plane bending
K	$\min \begin{cases} (16 + 1.2\gamma)\beta^{1.2}Q_g \\ 40\beta^{1.2}Q_g \end{cases}$		$(5 + 0.7\gamma)\beta^{1.2}$	$2.5 + (4.5 + 0.2\gamma)\beta^{2.6}$
Y	30β	$\min \begin{cases} 2.8 + (20 + 0.8\gamma)\beta^{1.6} \\ 2.8 + 36\beta^{1.6} \end{cases}$		
X	$6.4\gamma^{(0.6\beta^2)}$	$(2.8 + (12 + 0.1\gamma)\beta)Q_\beta$		

The following notes apply to Table 6-3:

(a) Q_β is a geometric factor defined by:

$$Q_\beta = \frac{0.3}{\beta(1 - 0.833\beta)} \quad \text{for } \beta > 0.6$$

$$Q_\beta = 1.0 \quad \text{for } \beta \leq 0.6$$

(b) Q_g is a gap factor defined by:

$$Q_g = 1 + 0.2 \left(1 - \frac{2.8g}{D} \right)^3 \quad \text{for } \frac{g}{D} \geq 0.05, \text{ but } Q_g \geq 1.0$$

$$Q_g = 0.13 + 0.65\phi\gamma^{0.5} \quad \text{for } \frac{g}{D} \leq -0.05$$

$$\text{where } \phi = \frac{tf_{y,b}}{Tf_{y,c}}$$

$f_{y,b}$ = yield strength of brace (or brace stub if present)

$f_{y,c}$ = yield strength of chord (or chord can if present)

Q_g = linear interpolated value between the limiting values of the above expressions for

$$-0.05 \leq \frac{g}{D} \leq 0.05$$

6.4.3.4 Chord action factor Q_f

Q_f is a design factor to account for the presence of factored actions in the chord.

$$Q_f = 1.0 + C_1 \frac{\sigma_{a,Sd}}{f_y} - C_2 \frac{\sigma_{my,Sd}}{1.62 f_y} - C_3 A^2 \quad (6.54)$$

The parameter A is defined as follows:

$$A^2 = \left(\frac{\sigma_{a,Sd}}{f_y} \right)^2 + \left(\frac{\sigma_{my,Sd}^2 + \sigma_{mz,Sd}^2}{1.62 f_y^2} \right) \quad (6.55)$$

where

- $\sigma_{a,Sd}$ = design axial stress in chord, positive in tension
- $\sigma_{my,Sd}$ = design in-plane bending stress in chord, positive for compression in the joint footprint
- $\sigma_{mz,Sd}$ = design out-of-plane bending stress in chord
- f_y = yield strength
- C_1, C_2, C_3 = coefficients depending on joint and load type as given in Table 6-4

Table 6-4 Values for C_1 , C_2 and C_3

Joint type	C_1	C_2	C_3
K joints under balanced axial loading	0.2	0.2	0.3
T/Y joints under brace axial loading	0.3	0	0.8
X joints under brace axial tension loading 1)			
$\beta \leq 0.9$	0	0	0.4
$\beta = 1.0$	0.2	0	0.2
X joints under brace axial compression loading 1)			
$\beta \leq 0.9$	0.2	0	0.5
$\beta = 1.0$	-0.2	0	0.2
All joints under brace moment loading	0.2	0	0.4

1) Linear interpolated values between $\beta = 0.9$ and $\beta = 1.0$

The average of the chord loads and bending moments on either side of the brace intersection should be used in Equations (6.54) and (6.55). The chord thickness at the joint should be used in the above calculations.

6.4.3.5 Design axial resistance for X and Y joints with joint cans

For Y and X joints with axial force and where a joint can is specified, the joint design resistance should be calculated as follows:

$$N_{Rd} = \left(r + (1-r) \left(\frac{T_n}{T_c} \right)^2 \right) N_{can,Rd} \quad (6.56)$$

where

- $N_{can,Rd}$ = N_{Rd} from Equation (6.52) based on chord can geometric and material properties, including Q_f calculated with respect to chord can
- T_n = nominal chord member thickness
- T_c = chord can thickness
- r = $L_c/2.5 D$ for joints with $\beta \leq 0.9$
 = $\frac{(4\beta-3) \cdot L_c}{1.5 D}$ for joints with $\beta > 0.9$
- L_c = effective total length. Figure 6-7 gives examples of calculation of L_c

In no case shall r be taken as greater than unity.

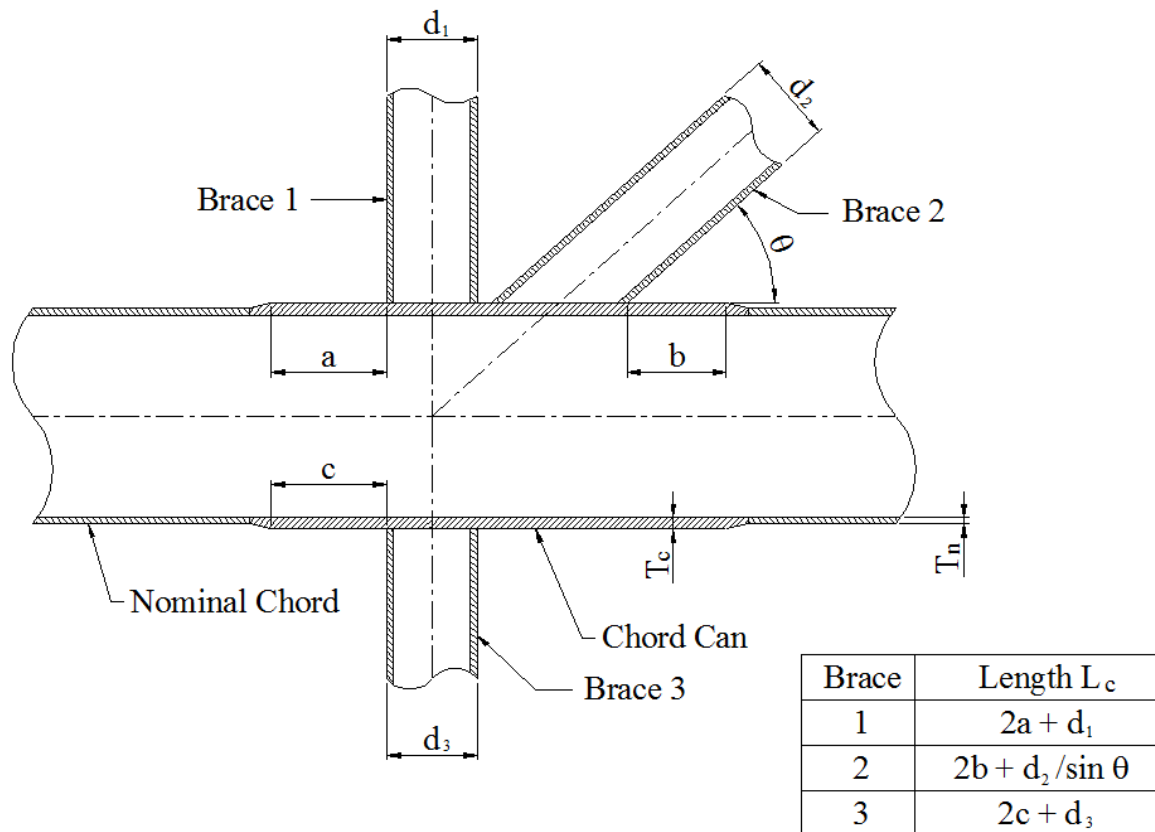


Figure 6-7 Examples of chord length L_c

6.4.3.6 Strength check

Joint resistance shall satisfy the following interaction equation for axial force and/or bending moments in the brace:

$$\frac{N_{Sd}}{N_{Rd}} + \left(\frac{M_{y,Sd}}{M_{y,Rd}} \right)^2 + \frac{M_{z,Sd}}{M_{z,Rd}} \leq 1 \quad (6.57)$$

where

N_{Sd}	=	design axial force in the brace member
N_{Rd}	=	the joint design axial resistance
$M_{y,Sd}$	=	design in-plane bending moment in the brace member
$M_{z,Sd}$	=	design out-of-plane bending moment in the brace member
$M_{y,Rd}$	=	design in-plane bending resistance
$M_{z,Rd}$	=	design out-of-plane bending resistance

6.4.4 Overlap joints

Braces that overlap in- or out-of-plane at the chord member form overlap joints.

Joints that have in-plane overlap involving two or more braces may be designed using the simple joint provision of 6.4.3 with the following exemptions and additions:

- Shear parallel to the chord face is a potential failure mode and should be checked.
- 6.4.3.5 does not apply to overlapping joints.
- If axial forces in the overlapping and through braces have the same sign, the combined axial force representing that in the through brace plus a portion of the overlapping brace forces should be used to check the through brace intersection capacity. The portion of the overlapping brace force can be

calculated as the ratio of cross sectional area of the brace that bears onto the through brace to the full area.

- d. For both in-plane or out-of-plane bending moments, the combined moment of the overlapping and through braces should be used to check the through brace intersection capacity. This combined moment should account for the sign of the moments. The overlapping brace should also be checked on the basis of the chord having through brace properties. Further, through brace capacity shall be checked for combined axial and moment loading in the overlapping brace. In this instance the Q_f associated with the through brace shall be used.

Joints with out-of-plane overlap may be assessed on the same general basis as in-plane overlapping joints, except that axial force resistance should normally revert to that for Y joints.

6.4.5 Ringstiffened joints

Design resistance of ringstiffened joints shall be determined by use of recognised engineering methods. Such methods can be elastic analyses, plastic analyses or linear or non-linear finite element analyses. See Clause 12.

6.4.6 Cast joints

Cast joints are defined as joints formed using a casting process. They can be of any geometry and of variable wall thickness.

The design of a cast joint requires calibrated FE analyses. An acceptable design approach for strength is to

limit the stresses found by linear elastic analyses to the design strength $\frac{f_y}{\gamma_M}$ using appropriate yield criteria.

Elastic peak stresses may be reduced following similar design principles as described in Clause 12, Comm. 6.4.5.

6.5 Strength of conical transitions

6.5.1 General

The provisions given in this section are for the design of concentric cone frusta between tubular sections. They may also be applied to conical transitions at brace ends, where the junction provisions apply only to the brace-end transition away from the joint.

6.5.2 Design stresses

6.5.2.1 Equivalent design axial stress in the cone section.

The equivalent design axial stress (meridional stress) at any section within the conical transition can be determined by the following equations:

$$\sigma_{\text{equ,Sd}} = \frac{\sigma_{\text{ac,Sd}} + \sigma_{\text{mc,Sd}}}{\cos \alpha} \quad (6.58)$$

$$\sigma_{\text{ac,Sd}} = \frac{N_{\text{Sd}}}{\pi(D_s - t_c \cos \alpha)t_c} \quad (6.59)$$

$$\sigma_{\text{mc,Sd}} = \frac{M_{\text{Sd}}}{\frac{\pi}{4}(D_s - t_c \cos \alpha)^2 t_c} \quad (6.60)$$

where

$\sigma_{\text{equ,Sd}}$ = equivalent design axial stress within the conical transition
 $\sigma_{\text{ac,Sd}}$ = design axial stress at the section within the cone due to global actions

$\sigma_{mc,Sd}$	=	design bending stress at the section within the cone due to global actions
D_s	=	outer cone diameter at the section under consideration
t_c	=	cone thickness (to be equal or larger than the thickness of the smaller tubular)
α	=	the slope angle of the cone (see Figure 6-8)

N_{Sd} and M_{Sd} are the design axial force and design bending moment at the section under consideration. $\sigma_{ac,Sd}$ and $\sigma_{mc,Sd}$ should be calculated at the junctions of the cone sides.

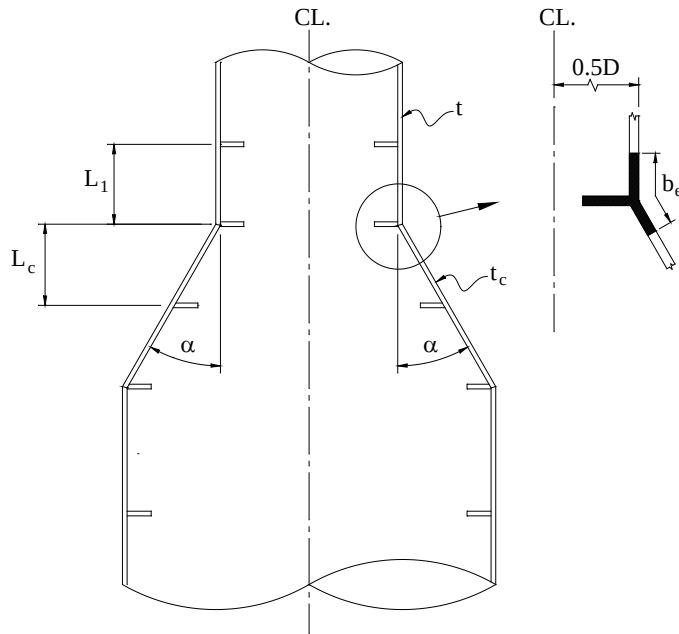


Figure 6-8 Cone geometry

6.5.2.2 Local bending stress at unstiffened junctions

In lieu of a detailed analysis, the local bending stress at unstiffened tubular-cone junctions can be estimated by the following equation:

$$\sigma_{ml,Sd} = 0.85 \sqrt{\frac{D_j}{t}} (\sigma_{at,Sd} + \sigma_{mt,Sd}) \tan \alpha \quad (6.61)$$

$\sigma_{ml,Sd}$	=	local design bending stress
D_j	=	cylinder diameter at junction
t	=	tubular member wall thickness
$\sigma_{at,Sd}$	=	design axial stress in tubular section at junction due to global actions
$\sigma_{mt,Sd}$	=	design bending stress in tubular section at junction due to global actions

6.5.2.3 Hoop stress at unstiffened junctions

The design hoop stress at unstiffened tubular-cone junctions due to unbalanced radial line forces may be estimated from:

$$\sigma_{hc,Sd} = 0.45 \sqrt{\frac{D_j}{t}} (\sigma_{at,Sd} + \sigma_{mt,Sd}) \tan \alpha \quad (6.62)$$

At the smaller-diameter junction, the hoop stress is tensile (or compressive) when $(\sigma_{at,Sd} + \sigma_{mt,Sd})$ is tensile (or compressive). Similarly, the hoop stress at the larger-diameter junction is tensile (or compressive) when $(\sigma_{at,Sd} + \sigma_{mt,Sd})$ is compressive (or tensile).

6.5.3 Strength requirements without external hydrostatic pressure

6.5.3.1 Local buckling under axial compression

For local buckling under combined axial compression and bending, Equation (6.63) should be satisfied at all sections within the conical transition.

$$\sigma_{equ,Sd} \leq \frac{f_{clc}}{\gamma_M} \quad (6.63)$$

where

$$f_{clc} = \text{local buckling strength of conical transition}$$

For conical transitions with slope angle $\alpha < 30^\circ$, f_{clc} can be determined using Equation (6.6) to Equation (6.8) with an equivalent diameter, D_e , at the section under consideration.

$$D_e = \frac{D_s}{\cos \alpha} \quad (6.64)$$

For conical transitions of constant wall thickness, it would be conservative to use the diameter at the larger end of the cone as D_s in Equation (6.64).

6.5.3.2 Junction yielding

Yielding at a junction of a cone should be checked on both tubular and cone sides. This section only applies when the hoop stress, $\sigma_{hc,Sd}$, is tensile.

For net axial tension, that is, when $\sigma_{equ,Sd}$ is tensile,

$$\sqrt{\sigma_{equ,Sd}^2 + \sigma_{hc,Sd}^2 - \sigma_{hc,Sd} \sigma_{equ,Sd}} \leq \frac{f_y}{\gamma_M} \quad \text{but} \quad \sigma_{equ,Sd} \leq \frac{f_y}{\gamma_M} \quad (6.65)$$

For net axial compression, that is, when $\sigma_{equ,Sd}$ is compression,

$$\sqrt{\sigma_{equ,Sd}^2 + \sigma_{hc,Sd}^2 + \sigma_{hc,Sd} |\sigma_{equ,Sd}|} \leq \frac{f_y}{\gamma_M} \quad (6.66)$$

where

$$\begin{aligned} \sigma_{equ,Sd} &= \sigma_{at,Sd} + \sigma_{mt,Sd} \text{ for checking stresses on the tubular side of the junction} \\ &= \frac{\sigma_{ac,Sd} + \sigma_{mc,Sd}}{\cos \alpha} \text{ for checking stresses on the cone side of the junction} \\ f_y &= \text{corresponding tubular or cone yield strength.} \end{aligned}$$

The local bending stress $\sigma_{mt,Sd}$ at a junction of a cone should be limited according to the following for $\sigma_{equ,Sd}$ at both sides of the junction:

$$\sigma_{ml,Sd} \leq 1.5 \frac{f_y}{\gamma_M} \left(1 - \frac{\sigma_{equ,Sd}^2}{f_y^2} \right) \quad (6.67)$$

γ_M = see 6.3.7 using N_{Sd} , $M_{y,Sd}$ and $M_{z,Sd}$ and geometry for the part with the largest D/t ratio.

6.5.3.3 Junction buckling

This section only applies when the hoop stress, $\sigma_{hc,Sd}$, is compressive. In the equations, $\sigma_{hc,Sd}$ denotes the positive absolute value of the hoop compression.

For net axial tension, that is, when $\sigma_{equ,Sd}$ is tensile,

$$a^2 + b^{2\eta} + 2vab \leq 1.0 \quad (6.68)$$

where:

$$a = \frac{\sigma_{equ,Sd}}{\frac{f_y}{\gamma_M}} \quad (6.69)$$

$$b = \frac{\sigma_{hc,Sd}}{\frac{f_{hj}}{\gamma_M}} \quad (6.70)$$

and v is Poisson's ratio of 0.3 and η is defined in Equation (6.38)

For net axial compression, that is, when $\sigma_{equ,Sd}$ is compressive,

$$\sigma_{equ,Sd} \leq \frac{f_{clj}}{\gamma_M} \quad (6.71)$$

and

$$\sigma_{hc,Sd} \leq \frac{f_{hj}}{\gamma_M} \quad (6.72)$$

where

f_{clj} = corresponding tubular or cone characteristic axial local compressive strength

f_{hj} can be determined using Equation (6.17) to Equation (6.19) with $f_{he} = 0.40E t/D_j$ and corresponding f_y .

The local bending stress $\sigma_{mlt,Sd}$ at a junction of a cone should be limited according to the following for $\sigma_{equ,Sd}$ at both sides of the junction:

$$\sigma_{ml,Sd} \leq 1.5 \frac{f_y}{\gamma_M} \left(1 - \frac{\sigma_{equ,Sd}^2}{f_y^2} \right) \quad (6.73)$$

γ_M = see 6.3.7 using N_{Sd} , $M_{y,Sd}$ and $M_{z,Sd}$ and geometry for the part with the largest D/t ratio.

6.5.4 Strength requirements with external hydrostatic pressure

6.5.4.1 Hoop buckling

Un stiffened conical transitions, or cone segments between stiffening rings with slope angle $\alpha < 30^\circ$, may be designed for hoop collapse by consideration of an equivalent tubular using Equation (6.44) or Equation (6.51) as appropriate. The effective diameter is $D/\cos\alpha$, where D is the diameter at the larger end of the cone segment. The equivalent nominal axial stress should be used to represent the axial stress in the design. The length of the cone should be the maximum distance between adjacent rings for ring-stiffened cone transitions or the maximum unstiffened distance, including the prismatic members on both ends of the cone, for an unstiffened cone transition. Note that the prismatic members on both ends of a ring-stiffened cone should be checked against the stipulations in 6.3.9.

6.5.4.2 Junction yielding and buckling

The net design hoop stress at a tubular-cone junction is given by the algebraic sum of $\sigma_{hc,Sd}$ and $\sigma_{h,Sd}$, that is

$$\sigma_{hj,Sd} = \sigma_{hc,Sd} + \sigma_{h,Sd} \quad (6.74)$$

where

$\sigma_{h,Sd}$ = design hoop stress due to the external hydrostatic pressure, see eq. (6.16)

When $\sigma_{hj,Sd}$ is tensile, the equations in 6.5.3.2 should be satisfied by using $\sigma_{hj,Sd}$ instead of $\sigma_{hc,Sd}$. When $\sigma_{hj,Sd}$ is compressive, the equations in 6.5.3.3 should be satisfied by using $\sigma_{hj,Sd}$ instead of $\sigma_{hc,Sd}$.

6.5.5 Ring design

6.5.5.1 General

A tubular-cone junction that does not satisfy the above criteria may be strengthened either by increasing the tubular and cone thicknesses at the junction, or by providing a stiffening ring at the junction.

6.5.5.2 Junction rings without external hydrostatic pressure

If stiffening rings are required, the section properties should be chosen to satisfy both the following requirements:

$$A_c = \frac{tD_j}{f_y} (\sigma_{a,Sd} + \sigma_{m,Sd}) \tan\alpha \quad (6.75)$$

$$I_c = \frac{tD_j D_c^2}{8E} (\sigma_{a,Sd} + \sigma_{m,Sd}) \tan\alpha \quad (6.76)$$

where

$\sigma_{a,Sd}$ = larger of $\sigma_{at,Sd}$ and $\sigma_{ac,Sd}$.
 $\sigma_{m,Sd}$ = larger of $\sigma_{mt,Sd}$ and $\sigma_{mc,Sd}$.
 D_c = diameter to centroid of composite ring section. See Note 4 in 6.3.6.2

A_c = cross-sectional area of composite ring section
 I_c = moment of inertia of composite ring section

In computing A_c and I_c , the effective width of shell wall acting as a flange for the composite ring section may be computed from:

$$b_e = 0.55(\sqrt{D_j t} + \sqrt{D_j t_c}) \quad (6.77)$$

Notes:

1. For internal rings, D_j should be used instead of D_c in Equation (6.76)
2. For external rings, D_j in Equation (6.75) and Equation (6.76) should be taken to the centroid of the composite ring.

6.5.5.3 Junction rings with external hydrostatic pressure

Circumferential stiffening rings required at the tubular-cone junctions should be designed such that the moment of inertia of the composite ring section is equal to or greater than the sum of Equation (6.76) and Equation (6.79):

$$I_{cT} \geq I_c + I_{ch} \quad (6.78)$$

where

$$I_{ch} = \frac{D_j^2}{16E} \left\{ t L_1 f_{he} + \frac{t_c L_c f_{hec}}{\cos^2 \alpha} \right\} \quad (6.79)$$

where

I_{cT} = moment of inertia of composite ring section with external hydrostatic pressure and with effective width of flange computed from Equation (6.77)
 D_j = diameter of tubular at junction. See Note 4 in 6.3.6.2
 L_c = distance to first stiffening ring in cone section along cone axis $\leq 1.13 \sqrt{\frac{D_e^3}{t}}$
 L_1 = distance to first stiffening ring in tubular section $\leq 1.13 \sqrt{\frac{D_j^3}{t}}$
 f_{he} = elastic hoop buckling strength for tubular
 f_{hec} = f_{he} for cone section treated as an equivalent tubular
 D_e = larger of equivalent diameters at the junctions

Notes:

3. A junction ring is not required for hydrostatic collapse if Equation (6.15) is satisfied with f_{he} computed using C_h equal to $0.44 (\cos \alpha) (t/D_j)$ in Equation (6.20), where D_j is the tubular diameter at the junction.
4. For external rings, D_j in Equation (6.79) should be taken to the centroid of the composite ring, except in the calculation of L_1 .

6.5.5.4 Intermediate stiffening rings

If required, circumferential stiffening rings within cone transitions may be designed using Equation (6.21) with an equivalent diameter equal to $D_s / \cos \alpha$, where D_s is the cone diameter at the section under consideration, t is the cone thickness, L is the average distance to adjacent rings along the cone axis and f_{he} is the average of the elastic hoop buckling strength values computed for the two adjacent bays.

6.6 Design of plated structures

Design of plated structures such as stiffened panels shall be done according to Part 1 of DNV-RP-C201 /14/. Design of plates may also be done according to NS-EN-1993-1-5 for cases where these standards give recommendations e.g. web of plate girders. Capacity checks according to this standard may imply utilisation of the plate into the post linear range and their capacity against dynamic loads should be considered. See also sub-clause 6.1

6.7 Design of cylindrical shells

Unstiffened and ring stiffened cylindrical shells subjected to axial force, bending moment and hydrostatic pressure may be designed according to 6.3. For more refined analysis of cylindrical shells or cylindrical shells with other stiffening geometry or loading, DNV-RP-C202 /13/ shall be used.

The susceptibility to less favourable post-critical behaviour associated with more slender geometry and more and /or larger stress components may be expressed through the reduced slenderness parameter, $\bar{\lambda}_s$, which is defined in DNV-RP-C202 /13/.

The resulting material factor for design of shell structures is given as

$$\begin{aligned}\gamma_M &= 1.15 && \text{for } \bar{\lambda}_s < 0.5 \\ \gamma_M &= 0.85 + 0.60\bar{\lambda}_s && \text{for } 0.5 \leq \bar{\lambda}_s \leq 1.0 \\ \gamma_M &= 1.45 && \text{for } \bar{\lambda}_s > 1.0\end{aligned}\tag{6.80}$$

$\bar{\lambda}_s$ is defined in DNV-RP-C202 /13/.

6.8 Design against unstable fracture

6.8.1 General

Normally brittle fracture in offshore structures is avoided by selecting materials according to Clause 5 and with only acceptable defects present in the structure after fabrication.

Unstable fracture may occur under unfavourable combinations of geometry, fracture toughness, welding defects and stress levels. The risk of unstable fracture is generally greatest with large material thickness where the state of deformation is plane strain. For normal steel qualities, this typically implies a material thickness in excess of 40 mm to 50 mm, but this is dependent on geometry, fracture toughness, weld defects and stress level.

See /8/ for guidance on the use of fracture mechanics. If relevant fracture toughness data is lacking, material testing should be performed.

6.8.2 Determination of maximum acceptable defect size

The design stress shall be determined with load coefficients given in NORSOK N-001. The maximum applied tensile stress, accounting also for possible stress concentrations, shall be considered when calculating the tolerable defect size. Relevant residual stresses shall be included in the evaluation. Normally, a structure is designed based on the principle that plastic hinges may develop without giving rise to unstable fracture. In such case, the design nominal stress for unstable fracture shall not be less than the yield stress of the member.

The characteristic fracture toughness, K_{Ic} , (or, alternatively K_c , J_c , J_{Ic} , $CTOD_c$), shall be determined as the lower 5% fractile of the test results. The design fracture toughness shall be calculated from Equation (6.81).

$$K_{Icd} = \frac{K_{Ic}}{\gamma_M}\tag{6.81}$$

where

$$\gamma_M = 1.15 \text{ for members where failure will be without substantial consequences}$$

$\gamma_M = 1.4$ for members with substantial consequences

Notes:

1. "Substantial consequences" in this context means that failure of the joint will entail:
Danger of loss of human life;
Significant pollution;
Major financial consequences.
2. "Without substantial consequences" is understood failure where it can be demonstrated that the structure satisfy the requirement to damaged condition according to the Accidental Limit States with failure in the actual joint as the defined damage.

The design values of the J-integral and CTOD shall be determined with a safety level corresponding to that used to determine the design fracture toughness. For example, the design CTOD is found from the following formula:

$$CTOD_{cd} = \frac{CTOD_c}{\gamma_M^2} \quad (6.82)$$

The maximum defect size likely to remain undetected (a_i) shall be established, based on an evaluation of the inspection method, access for inspection during fabrication, fabrication method, and the thickness and geometry of the structure,. When determining the value of a_i consideration shall be given to the capabilities of the inspection method to detect, localise, and size the defect.

The maximum allowable defect size (a_m), shall be calculated on the basis of the total stress (or corresponding strains) and the design fracture toughness. It shall be shown that $a_i < a_m$.

For a structure subjected to fatigue loading, the crack growth may be calculated by fracture mechanics. The initial defect size shall be taken as a_i . The final crack size, a_u , shall be determined with the fatigue load applied over the expected life time. It shall be verified that $a_u < a_m$.

7 SERVICEABILITY LIMIT STATES

General requirements for the serviceability limit states are given in NORSOK N-001.

8 FATIGUE LIMIT STATES

8.1 General

In this NORSOK standard requirements are given in relation to fatigue analyses based on fatigue tests and fracture mechanics. Reference is made to DNV-RP-C203 /15/for more details with respect to fatigue design.

The aim of fatigue design is to ensure that the structure has an adequate fatigue life. Calculated fatigue lives can also form the basis for efficient inspection programmes during fabrication and the operational life of the structure.

The design fatigue life for the structure components should be based on the structure service life specified by the operator. If no structure service life is specified by the operator, a service life of 15 years shall be used. A short design fatigue life will imply shorter inspection intervals.

To ensure that the structure will fulfil the intended function, a fatigue assessment, supported where appropriate by a detailed fatigue analysis should be carried out for each individual member which is subjected to fatigue loading. It should be noted that any element or member of the structure, every welded joint and attachment, or other form of stress concentration, is potentially a source of fatigue cracking and should be individually considered.

The number of load cycles shall be multiplied with the appropriate factor in Table 8-1 before the fatigue analysis is performed.

Table 8-1 Design fatigue factors

Classification of structural components based on damage consequence	Not accessible for inspection and repair or in the splash zone	Accessible for inspection, maintenance and repair, and where inspections or maintenance is planned	
		Below splash zone	Above splash zone
Substantial consequences	10	3	2
Without substantial consequences	3	2	1

"Substantial consequences" in this context means that failure of the joint will entail

- a) danger of loss of human life;
- b) significant pollution;
- c) major financial consequences.

"Without substantial consequences" is understood failure where it can be demonstrated that the structure satisfy the requirement to damaged condition according to the ALSs with failure in the actual joint as the defined damage.

Welds in joints below 150 m water depth should be assumed inaccessible for in-service inspection.

In project phases where it is possible to increase fatigue life by modification of structural details, grinding of welds should not be assumed to provide a measurable increase in the fatigue life.

8.2 Methods for fatigue analysis

The fatigue analysis should be based on S-N data, determined by fatigue testing of the considered welded detail, and the linear damage hypothesis. When appropriate, the fatigue analysis may alternatively be based on fracture mechanics. If the fatigue life estimate based on fatigue tests is short for a component where a failure may lead to substantial consequences, a more accurate investigation considering a larger portion of the structure, or a fracture mechanics analysis, should be performed. For calculations based on fracture mechanics, it should be documented that the planned in-service inspections accommodate a sufficient time interval between time of crack detection and the time of unstable fracture. See also 6.8. Reference is made to /15/ for more details.

All significant stress ranges, which contribute to fatigue damage in the structure, should be considered. The long term distribution of stress ranges may be found by deterministic or spectral analysis. Dynamic effects shall be duly accounted for when establishing the stress history.

9 ACCIDENTAL DAMAGE LIMIT STATES

9.1 General

The accidental damage limit states should be checked for accidental loads and abnormal environmental loads (return period 10 000 year). See NORSOK N-003 for details.

The material factor γ_M is 1.0 for ALSs unless noted otherwise.

For tubular members and conical sections the material factor in ALS shall be taken as the value calculated from 6.3.7 but divided by 1.15.

The material factors according to Table 9-1 shall be used if NS-EN 1993-1-1 and NS-EN 1993-1-8 is used for calculation of structural resistance.

Table 9-1 Material factors

Type of calculation	Material factor 1)	Value
Resistance of Class 1,2 or 3 cross-sections	γ_{M0}	1.0
Resistance of Class 4 cross-sections	γ_{M1}	1.0
Resistance of member to buckling	γ_{M1}	1.0
Resistance of net section at bolt holes	γ_{M2}	1.1
Resistance of fillet and partial penetration welds	γ_2	1.1
Resistance of bolted connections	γ_2	1.1

1) Symbols according to NS-EN 1993-1-1 and NS-EN 1993-1-8.

For resistance check where other material factors are used than given in Table 9-1, the recommended material factor in NS-EN 1993-1-1, NS-EN 1993-1-5 and NS-EN 1993-1-8 should be multiplied by an additional building code material factor $\gamma_{BC} = 0.9$.

9.2 Check for accidental actions

The structure shall be checked for all ALSs for the design accidental actions defined in the risk analysis.

The structure shall according to NORSOK N-001 be checked in two steps:

- Resistance of the structure against design accidental actions
- Post accident resistance of the structure against environmental actions. Should only be checked if the resistance is reduced by structural damage caused by the design accidental actions

The overall objective of design against accidental actions is to achieve a system where the main safety functions are not impaired by the design accidental actions. In general the failure criteria to be considered, should also be defined in the risk analyses, see Clause 12.

The design against accidental actions may be done by direct calculation of the effects imposed by the actions on the structure, or indirectly, by design of the structure as tolerable to accidents. Examples of the latter are compartmentation of floating units which provides sufficient integrity to survive certain collision scenarios without further calculations.

The inherent uncertainty of the frequency and magnitude of the accidental loads, as well as the approximate nature of the methods for determination of accidental action effects, shall be recognised. It is therefore essential to apply sound engineering judgement and pragmatic evaluations in the design.

If non-linear, dynamic finite elements analysis is applied, it shall be verified that all behavioural effects and local failure modes (e.g. strain rate, local buckling, joint overloading, joint fracture) are accounted for implicitly by the modelling adopted, or else subjected to explicit evaluation.

Typical accidental actions are

- impact from ship collisions,
- impact from dropped objects,
- fire,
- explosions.

The different types of accidental actions require different methods and analyses to assess the structural resistance. Design recommendations for the most common types of accidental actions are given in Annex A.

10 REASSESSMENT OF STRUCTURES

10.1 General

Reassessment of existing structures should be made according to Norsok N-006

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12 COMMENTARY

This clause provides additional guidance and background to selected clauses of this NORSOK standard.

Comm. 1 Scope

In general the provisions of this NORSOK standard are developed, tested and calibrated for steel with traditional stress/strain relations. Design of structures made from steel material with higher yield strength may require additional or different design checks due to, among other the following effects:

9. - usually a higher yield to tensile strength ratio implying less strain hardening;
10. - larger elastic deflection before reaching resistance limit, which is important where second-order effects play a role, usually reduced weld overmatch leading to increased risk of failure in weld material;
11. - reduced maximum elongation.

Comm. 4 General provisions

The notation R_k and S_k shall be read as indicative for a resulting characteristic resistance and action effect respectively. In the general case the design resistance, R_d , will be a function of several parameters where the material parameter f_y should be divided by γ_M , and S_d will be derived as a summation of different characteristic actions multiplied with different partial coefficients.

The groups of limit states, applied in this NORSOK standard, are defined according to ISO 19900. In contrast, NS-ENV 1993 1-1 and most national building codes are treating limit states associated with failure due to accidental loads or fatigue as ultimate limit states. Whilst fatigue and failure from accidental loads are characteristically similar to ultimate limit states, it is convenient to distinguish between them due to different load factors for ULS, FLS and ALS. FLS and ALS may therefore be regarded as subgroups of ULS.

Comm. 5.1 Design class

The check for damaged condition according to the ALS imply that the structure with damage is checked for all characteristic actions, but with all safety factors set to unity. With regard to material selection the associated damage is brittle failure or lamellar tearing. If the structure subjected to one such failure is still capable of resisting the characteristic loads, the joint or component should be designated DC 3 or DC 4. As an example one can consider a system of four cantilever beams linked by a transverse beam at their free ends. A material failure at the supports in one of the beams will only reduce the resistance with 25 % (assuming sufficient strength in the transverse beam). Since the structure without damage need to be checked with partial safety factors according to ULSs, such a system will prove to satisfy ALS, and design class DC 3 or DC 4 dependent upon complexity will apply. A similar system of only two cantilever beams will normally not fulfill the ALS criterion to damage condition and DC 1 or DC 2 will apply.

Comm. 5.2 Steel quality level

Steel materials selected in accordance with the tabled SQL and as per MDS in NORSOK M-120 are assured to be of the same weldability within each SQL group, irrelevant of the actual material thickness or yield strength chosen. Requirements for SQL I are more severe than those for SQL II.

This is achieved through the different optional requirements stated in each MDS for each SQL. The achievement of balanced weldability is reached mainly by the following two means:

- requirement to higher energy absorption at toughness testing for yield strength above 400 MPa;
- lowering the test temperature for qualification testing, reflecting the differences between SQL II and I and stepwise increases in material thickness.

For high strength material of 420 MPa and above, the MDSs assume the same minimum yield strength irrelevant of material thickness of plates. This is normally achieved through adjustments in chemical composition, but without jeopardising the required weldability.

Improved properties in the through thickness direction (SQL I) should be specified for steel materials where failure due to lamellar tearing will mean significant loss of resistance. Alternatively the plate can be tested after welding to check that lamellar tearing has not taken place.

Comm. 6 Ultimate limit states

Comm. 6.1 General

NS-EN-1993-1-1 uses the following definition of ULSS: "ULSSs 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 ULSSs". For structures designed according to this NORSOK standard structural failures which will imply significant pollution or major financial consequences should be considered in addition to human safety.

All steel structures behave more or less non-linear when loaded to their ultimate limit. The formulae for design resistance in this NORSOK standard or similar codes and standards are therefore developed on the basis that permanent deformation may take place.

Traditionally, offshore structures are analysed by linear methods to determine the internal distribution of forces and moments, and the resistances of the cross-sections are checked according to design resistances found in design codes. The design formulae often require deformations well into the in-elastic range in order to mobilise the prescribed resistances. However, no further checks are considered necessary as long as the internal forces and moments are determined by linear methods.

When non-linear analysis methods are used, additional checks of ductility and repeated yielding, are to be performed. The check for repeated yielding is only necessary in case of cyclic loading, e.g. wave loads.

The check for ductility requires that all sections subjected to deformation into the in-elastic range should deform without loss of the assumed load-bearing resistance. Such loss of resistance can be due to fracture, instability of cross-sectional parts or member buckling. The design codes give little guidance on this issue, with exception for stability of cross-sectional parts in yield hinges, which will normally be covered by requirement to cross-sectional class 1, see e.g. NS-EN-1993-1-1.

The check against repeated yielding is often referred to as the check that the structure can achieve a stable state called shakedown. In the general case it is necessary to define a characteristic cycling load and to use this load with appropriate partial safety factors. It should be checked that yielding only take place in the first few loading cycles and that later load repetitions only cause responses in the linear range. Alternatively a low cycle fatigue check can be performed and substitute the check for shakedown.

If non-linear analyses are applied, it shall be checked that the analysis tool and the modelling adopted represent the non-linear behaviour of all structural elements that may contribute to the failure mechanism with sufficient accuracy, see /11/. Stiffness, resistance and post ultimate behaviour (if applicable) should be represented, including local failure modes such as local buckling, joint overload, joint fracture, etc.

Use of non-linear analysis methods may result in more structural elements being governed by the requirements to the SLS and additional SLS requirements may be needed compared with design using linear methods.

The following simplifications are valid for design with respect to the ultimate limit states, provided that ductile failure modes can be assumed:

1. - built in stresses from fabrication and erection may be neglected when fabrication requirements according to NORSOK M-101 are met;
2. - stresses from differential temperature under normal conditions (e.g. sun heating) may be neglected;
3. - the analytical static model need not in detail represent the real structure, e.g. secondary (deflection induced) moments may be neglected for trusses.

If the design is based on other parts of NS-EN-1993 than part 1-1- and 1-8 it is recommended to multiply the recommended material factor in the code if not given in Table 6-1 by an additional building code modification factor $\gamma_{BC} = 1.05$.

Comm. 6.2 Ductility

Steel structures behave generally ductile when loaded to their limits. The established design practise is based on this behaviour, which is beneficial both with respect to the design and performance of the structure. For a ductile structure, significant deflections may occur before failure and thus give a collapse warning. Ductile structures also have larger energy absorption capabilities against impact loads. The possibility for the structure to redistribute stresses lessens the need for an accurate stress calculation during design as the

structure may redistribute forces and moments to be in accordance with the assumed static model. This is the basis for use of linear analyses for ULS checks even for structures, which behave significantly non-linear when approaching their ultimate limit states.

Comm. 6.3.1 General

This subclause has been developed specifically for circular tubular shapes that are typical of offshore platform construction. The types of tubulars covered include fabricated roll-bent tubulars with a longitudinal weld seam, hot-finished seamless pipes, and ERW pipes that have undergone some types of post-weld heat treatment or normalisation to relieve residual stresses. Relief of the residual stresses is necessary to remove the “rounded” stress-strain characteristic that frequently arises from the ERW form of manufacture.

The limit of D/t applies to tubular members that are part of a frame system. The formulas may be used for more slender members if the member is not a part of a structural system, e.g. a tubular buoyancy tank.

The recommendations in this subclause are basically in accordance with ISO 19902.

Comm. 6.3.3 Axial compression

Tubular members subjected to axial compression are subject to failure due to either material yielding, overall column buckling, local buckling, or a combination of these failure modes.

The characteristic equation for column buckling is a function of $\bar{\lambda}$, a normalised form of column slenderness parameter given by $(f_{cl}/f_E)^{0.5}$; where f_{cl} is the local buckling strength of the cross-section and f_E is the Euler buckling strength for a perfect column.

For members with two or more different cross sections, the following steps can be used to determine the resistance:

1. Determine the elastic buckling load, N_E , for the complete member, taking into accounts the end restraints and variable cross-section properties. In most cases, the effective length factor of the member needs to be determined.
2. The design compressive resistance, $N_{c,Rd}$, is given by

$$N_{c,Rd} = \frac{N_{cl}}{\gamma_M} \left(1 - 0.28 \frac{N_{cl}}{N_E} \right) \quad \text{for } \frac{N_{cl}}{N_E} < 1.34 \quad (12.1)$$

$$N_{c,Rd} = \frac{0.9 N_E}{\gamma_M} \quad \text{for } \frac{N_{cl}}{N_E} \geq 1.34 \quad (12.2)$$

in which

N_{cl}	=	smallest characteristic local axial compressive strength of all the cross sections
	=	$f_{cl} A$
f_{cl}	=	as given by Equation (6.6) or Equation (6.7)
A	=	cross-sectional area

In design analysis, a member with variable cross sections can be modelled with several prismatic elements. For each prismatic element, added length and/or input effective length factor are used to ensure that the design compressive resistance is correctly determined.

The theoretical value of C_x for an ideal tubular is 0.6. However, a reduced value of $C_x = 0.3$ is recommended for use in Equation (6.8) to account for the effect of initial geometric imperfections within tolerance limits given in NORSOK M-101. A reduced value of $C_x = 0.3$ is also implicit in the limits for f_y/f_{cle} given in Equation (6.6) or Equation (6.7).

Short tubular members subjected to axial compression will fail either by material yielding or local buckling, depending on the diameter-to-thickness (D/t) ratio. Tubular members with low D/t ratios are generally not subject to local buckling under axial compression and can be designed on the basis of material yielding, i.e., the local buckling stress may be considered equal to the yield strength. However, as the D/t ratio increases, the elastic local buckling strength decreases, and the tubular should be checked for local buckling.

Un stiffened thin-walled tubular subjected to axial compression and bending are prone to sudden failures at loads well below the theoretical buckling loads predicted by classical small-deflection shell theory. There is a sudden drop in load-carrying resistance upon buckling of such members. The post-buckling reserve strength of tubular members is small, in contrast to the post-buckling behaviour of flat plates in compression, which usually continue to carry substantial load after local buckling. For this reason, there is a need for more conservatism in the definition of buckling load for tubulars than for most other structural elements. The large scatter in test data also necessitates a relatively conservative design procedure. The large scatter in test data is partly caused by initial imperfections generated by fabrication. Other factors of influence are boundary conditions and built-in residual stresses, (see /3/ and /4/).

Some experimental evidence indicates that inelastic local buckling may be less sensitive to initial imperfections and residual stresses than elastic local buckling, see /3/. Therefore, in order to achieve a robust design, it is recommended to select member geometry such that local buckling due to axial forces is avoided.

The characteristic equations are developed by screening test data and establishing the curve at 95 % success at the 50 % confidence level, which satisfies the following conditions:

- 1) it has a plateau of material characteristic yield strength over the range $0 \leq f_y/f_{cle} \leq 0.17$,
- 2) it has the general form of Equation (6.7),
- 3) it converges to the elastic critical buckling curve with increasing member slenderness ratio,
- 4) the difference between the mean minus 1.645 standard deviations of test data and the developed equations is minimum.

The local buckling data base contains 38 acceptable tests performed by several different investigators, see /3/.

A comparison between test data and the characteristic local buckling strength equation, Equation (6.6) to Equation (6.8) was made. The developed equations have the bias of 1.065, the standard deviation of 0.073, and the coefficient of variation of 0.068.

The elastic local buckling stress formula recommended in Equation (6.8) represent one-half of the theoretical local buckling stress computed using classical small-deflection theory. This reduction accounts for the detrimental effect of geometric imperfections. Based on the test data shown in /3/, this reduction is considered to be conservative for tubulars with $t \geq 6$ mm and $D/t < 120$. Offshore platform members typically fall within these dimensional limits. For thinner tubulars and tubulars with higher D/t ratios, larger imperfection reduction factors may be required, see /13/.

The local buckling database limits the applicability of the nominal strength equations to $D/t < 120$ and $t \geq 6$ mm. Reference /13/ provides guidance for the design of tubular members beyond these dimensional limits.

Comm. 6.3.6.1 Hoop buckling

Un-stiffened tubular members under external hydrostatic pressure are subject to elastic or inelastic local buckling of the shell wall between the restraints. Once initiated, the collapse will tend to flatten the member from one end to the other. Ring-stiffened members are subject to local buckling of the shell wall between rings. The shell buckles between the rings, while the rings remain essentially circular. However, the rings may rotate or warp out of their plane. Ring-stiffened tubular members are also subject to general instability, which occurs when the rings and shell wall buckle simultaneously at the critical load. It is desirable to provide rings with sufficient overstrength to prevent general instability. Reference is made to /14/. For tubular members satisfying the maximum out-of-roundness tolerance of 1 %, the hoop buckling strength is given by Equation (6.17) to Equation (6.19). For ring-stiffened members, Equation (6.17) to Equation (6.19) gives the hoop buckling strength of the shell wall between the rings. To account for the possible 1 % out-of roundness, the elastic hoop buckling stress is taken as 0.8 of the theoretical value calculated using classical small deflection theory. That is, $C_h = 0.44t/D$, whereas the theoretical $C_h = 0.55t/D$. In addition, the remaining C_h values are lower bound estimates.

For members with out-of-roundness greater than 1 %, but less than 3 %, a reduced elastic hoop buckling strength, f_{he} , should be determined, see /3/. The characteristic hoop buckling strength, f_h , is then determined using the reduced f_{he} .

$$\text{Reduced } f_{he} = \alpha \frac{f_{he}}{0.8}$$

where

$$\alpha = \text{geometric imperfection factor}$$

$$= 1 - 0.2 \sqrt{\frac{D_{\max} - D_{\min}}{0.01 D_{\text{nom}}}}$$

$$\frac{D_{\max} - D_{\min}}{0.01 D_{\text{nom}}} = \text{out-of-roundness (\%)}$$

where $D_{\max}$ and $D_{\min}$ are the maximum and minimum of any measured diameter at a cross section and D_{nom} the nominal diameter.

Comm. 6.3.6.2 Ring stiffener design

The formula recommended for determining the moment of inertia of stiffening rings, Equation (6.21), provides sufficient strength to resist buckling of the ring and shell even after the shell has buckled between stiffeners. It is assumed that the shell offers no support after buckling and transfers all its forces to the effective stiffener section. The stiffener ring is designed as an isolated ring that buckles into two waves ($n=2$) at a collapse pressure 20 % higher than the strength of the shell.

The effect of ring stiffeners on increased axial resistance for large diameter/thickness ratios is not accounted for in this section. Reference is made to /13/ for guidance on this.

Comm. 6.3.8 Tubular members subjected to combined loads without hydrostatic pressure

This subclause describes the background of the design requirements in 6.3.8, which covers un-stiffened and ring-stiffened cylindrical shell instability mode interactions when subjected to combined axial and bending loads without hydrostatic pressure

In this subclause and 6.3.9, the designer should include the second order frame moment or $P-\Delta$ effect in the bending stresses, when it is significant. The $P-\Delta$ effect may be significant in the design of un-braced deck legs, piles, and laterally flexible structures.

Comm. 6.3.8.1 Axial tension and bending

This subclause provides a resistance check for components under combined axial tensile load and bending. The interaction equation is modified compared with ISO/DIS 19902 as the term for axial load is raised in the power of 1.75.

Comm. 6.3.8.2 Axial compression and bending

This subclause provides an overall beam-column stability check, Equation (6.27), and strength check, Equation (6.28), for components under combined axial compression force and bending.

Use of ϕ -functions (see /5/) implies that exact solutions are obtained for the considered buckling problem. General effective buckling lengths have been derived using the ϕ -functions incorporating end flexibility of the members. The results for an X-brace with four equal-length members are shown in Figure 12-1 to Figure 12-3 as a function of the load distribution in the system Q/P and of the end rotational stiffness. P is the maximum compression force. The non-dimensional parameter ρ is given as:

$$\rho = \frac{CL}{EI}$$

where C is the local rotational stiffness at a node (accounting for local joint stiffness and stiffness of the other members going into the joint) and I is the moment of inertia of the tubular member. Normally L refers to center-line-to-center-line distances between nodes.

The results in Figure 12-2 indicates for realistic end conditions ($\rho > 3$) for single braces, i.e., when $Q/P = 1$, a k -factor less than 0.8 is acceptable provided end joint flexibilities are not lost as the braces become fully loaded. Experimental results relating to frames seem to confirm that 0.7 is acceptable. For X-braces where the magnitude of the tension brace load is at least 50 % of the magnitude of the compression brace load,

i.e., when $Q/P < -0.5$, and the joints remain effective, $k = 0.45$ times the length L is supported by these results, whilst 0.4 seems justified based on experimental evidence.

Figure 12-2 and Figure 12-3 provide effective length factors for X-braces when the longer segment is equal to 0.6 times the brace length and 0.7 times the brace length, respectively.

To estimate the effective length of a un-braced column, such as superstructure legs, the use of the alignment chart in Figure 12-4 provides a fairly rapid method for determining adequate k -values.

For cantilever tubular members, an independent and rational analysis is required in the determination of appropriate effective length factors. Such analysis shall take full account of all large deflection ($P-\Delta$) effects. For a cantilever tubular member, $C_m=1.0$.

The use of the moment reduction factor (C_m) in the combined interaction equations, such as Equation (6.27), is to obtain an equivalent moment that is less conservative. The C_m values recommended in Table 6-2 are similar to those recommended in /6/.

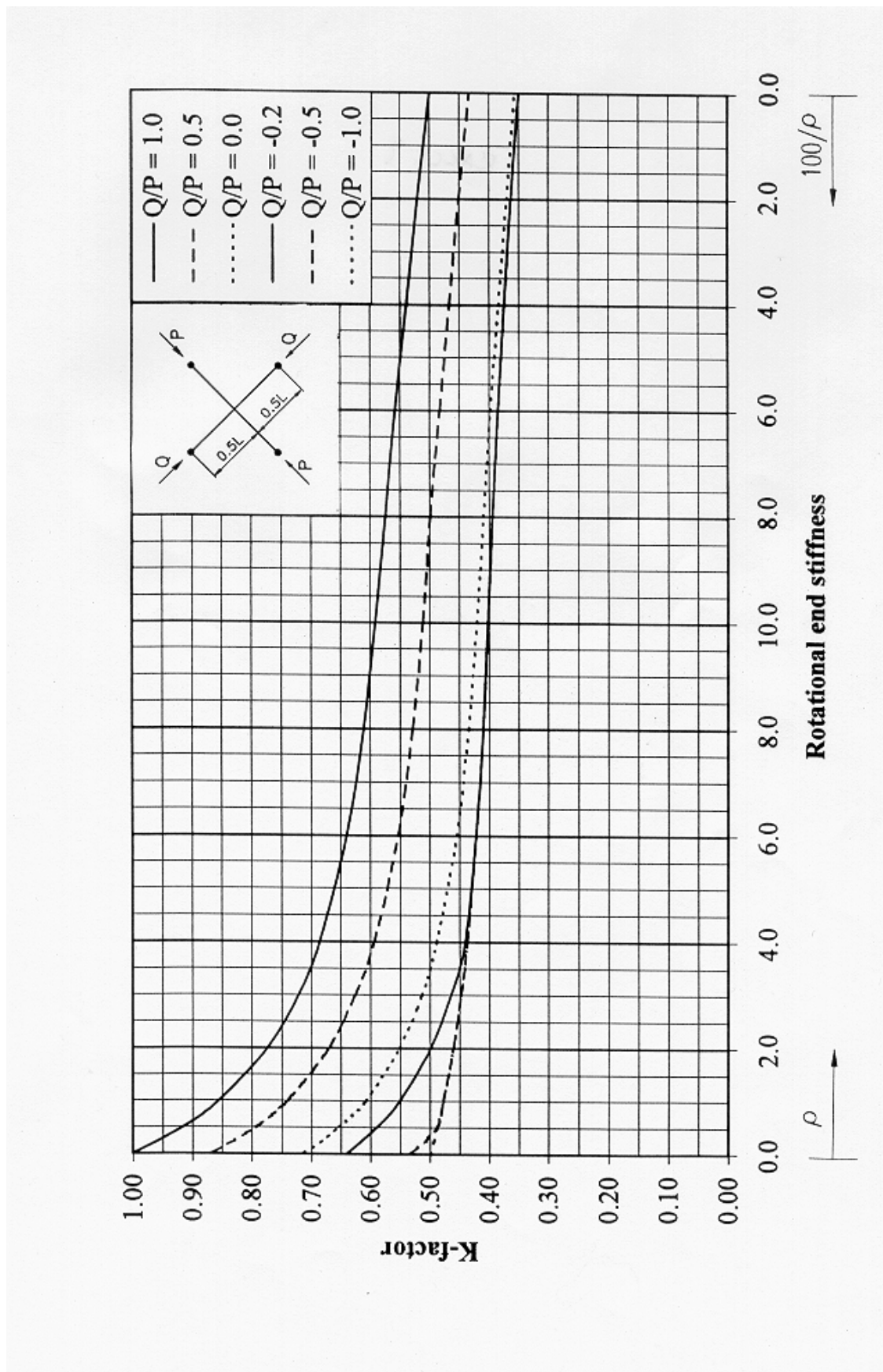


Figure 12-1 Effective length factors for a X-brace with equal brace lengths

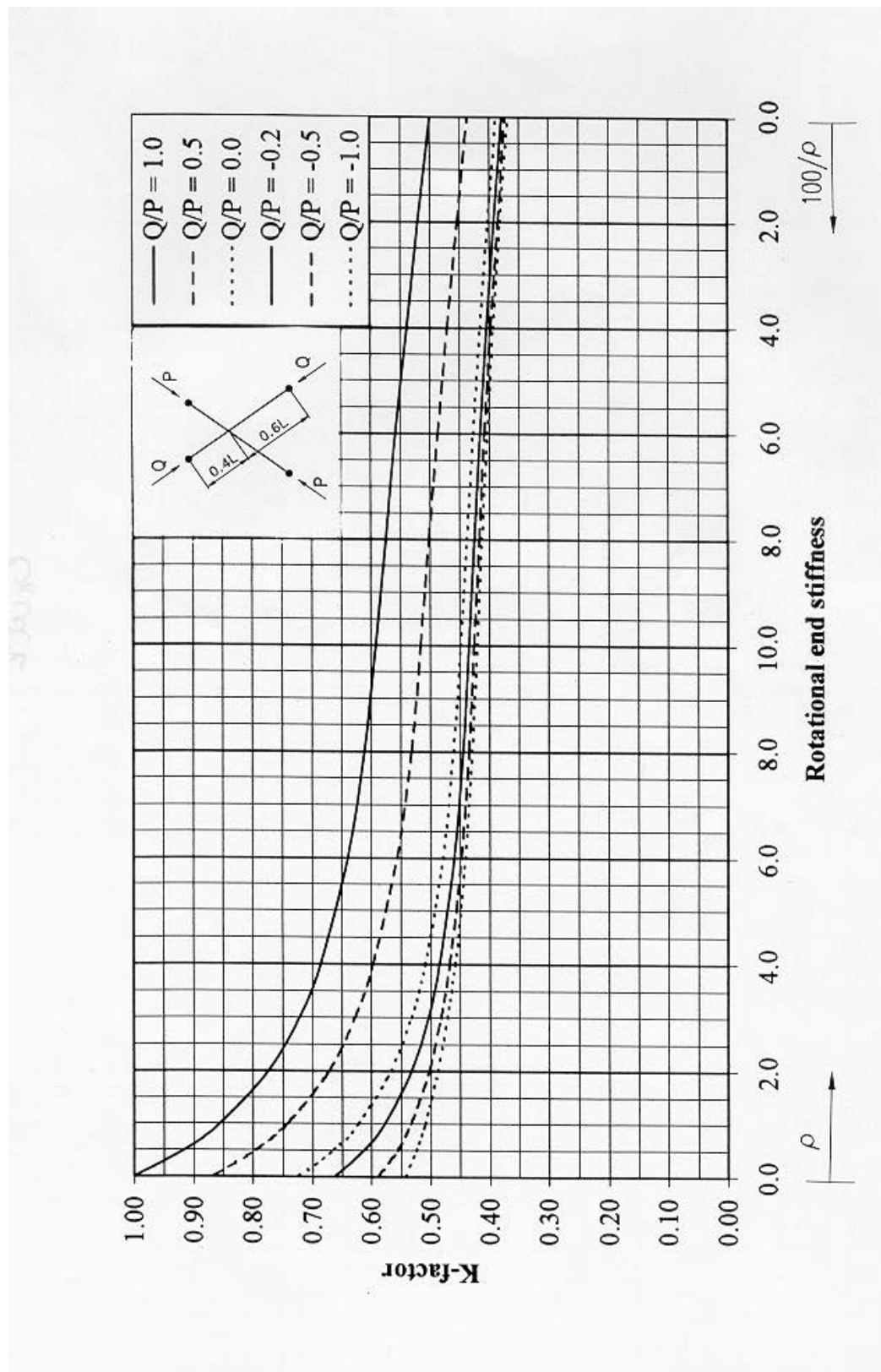


Figure 12-2 Effective length factors for a X-brace with the shorter segment equal to 0.4 times the brace length

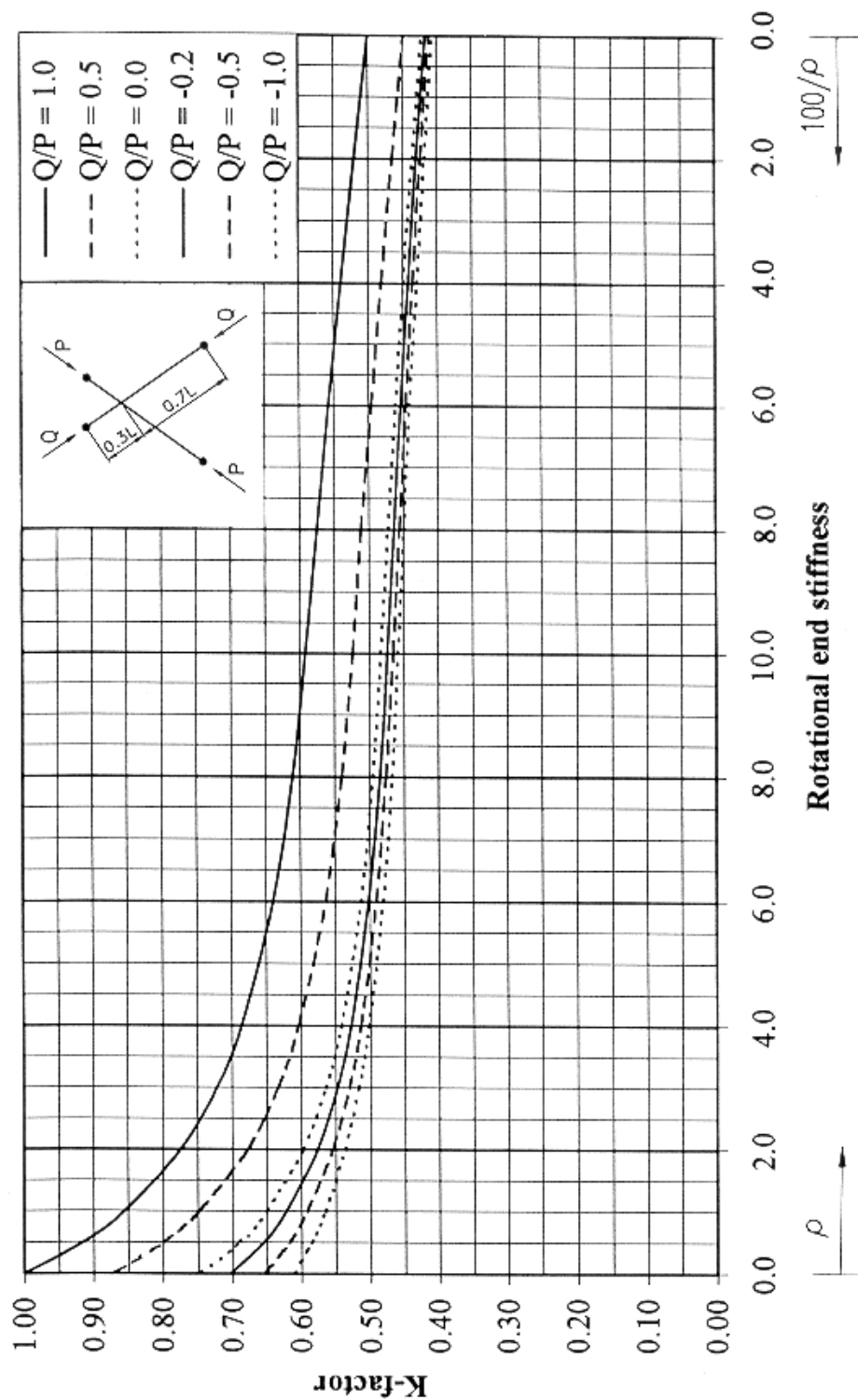


Figure 12-3 Effective length factors for a X-brace with the shorter segment equal to 0.3 times the brace length

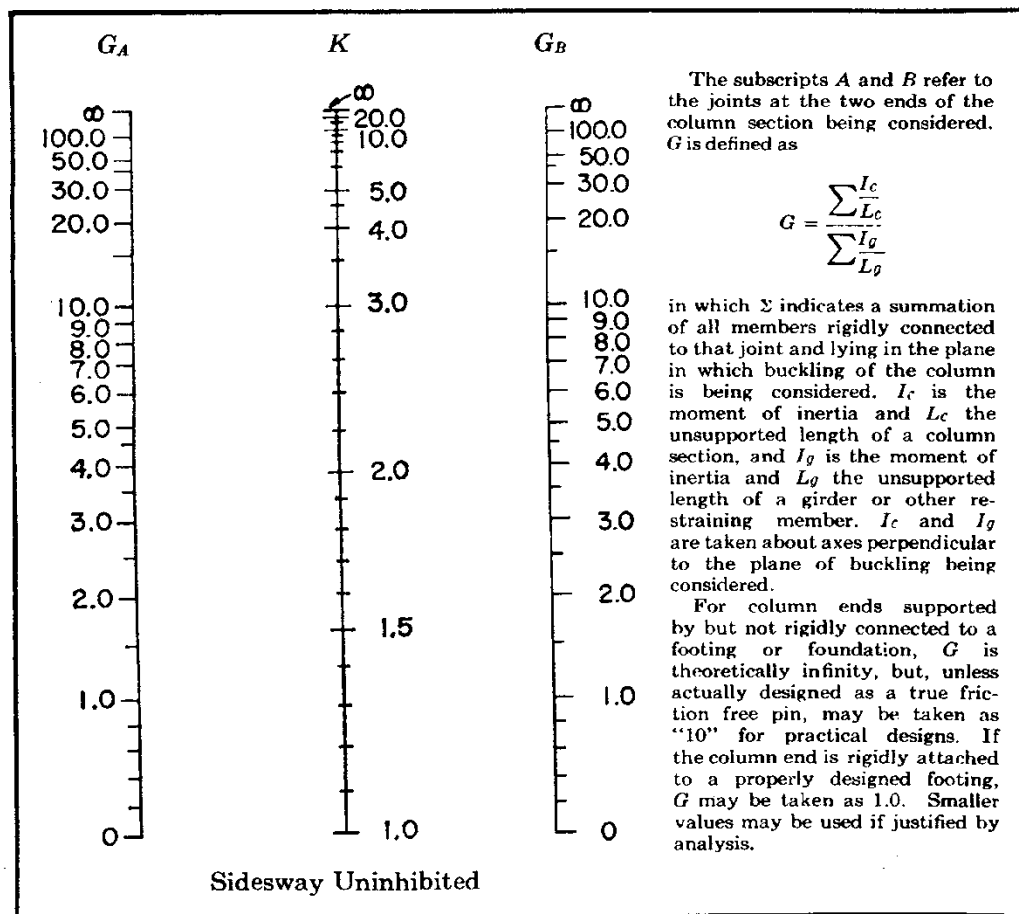


Figure 12-4 Alignment chart for effective length of columns in continuous frames

Comm. 6.3.9 Tubular members subjected to combined loads with hydrostatic pressure

This subclause provides strength design interaction equations for the cases in which a tubular member is subjected to axial tension or compression, and/or bending combined with external hydrostatic pressure.

Some guidance on significance of hydrostatic pressure may be found from Figure 12-5 for a given water depth and diameter/thickness ratio.

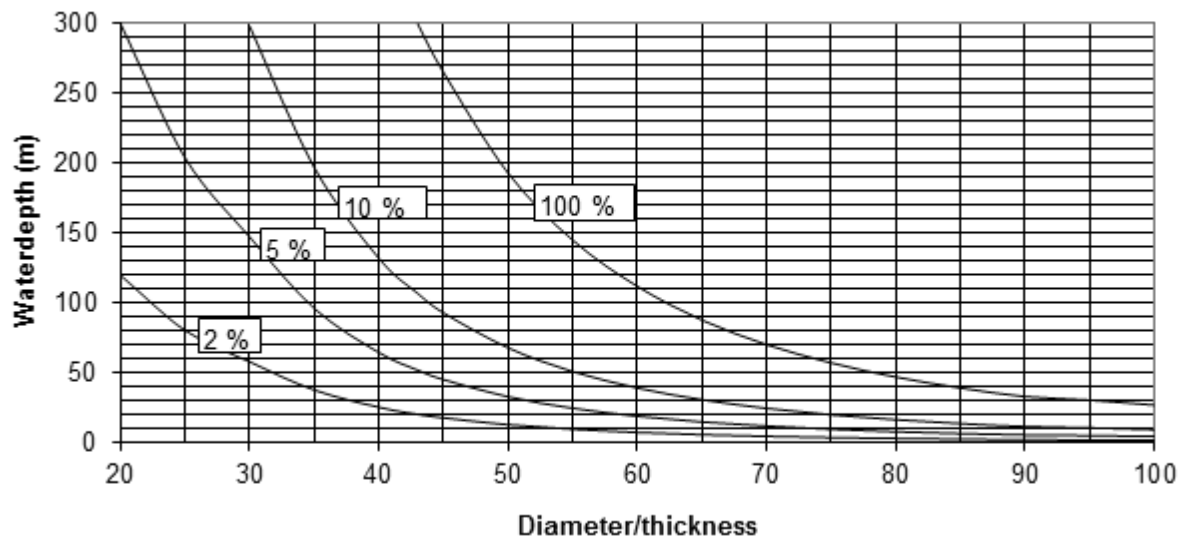


Figure 12-5 Reduction in bending resistance of members due to external pressure, ($\gamma_M \cdot \gamma_f = 1.5$, $f_y = 350$ MPa)

The design equations are categorised into two design approaches, Method A and Method B. The main purpose of providing two methods is to facilitate tubular member design by the two common analyses used by designers. Either design method is acceptable. In the limit when the hydrostatic pressure is zero, the design equations in this section reduce to those given in 6.3.8.

In both methods the hoop compression is not explicitly included in the analysis, but its effect on member design is considered within the design interaction equations. For both design methods, the hoop collapse design check stipulated in 6.3.6 shall be satisfied first.

Method A should be used when the capped-end axial compression due to external hydrostatic pressure is not explicitly included in the analysis, but its effect is accounted for while computing the member utilisation ratio.

Method B should be used when the capped-end axial compression due to external hydrostatic pressure is included explicitly in the analysis as nodal loads. The explicit application of the capped-end axial compression in the analysis allows for a more precise redistribution of the capped-end load based on the relative stiffness of the braces at a node.

The two methods are not identical. However, since redistribution of the capped-end axial compression in Method B is minimal because of similar brace sizes at a node, the difference between the two methods should be small.

Comm. 6.3.9.1 Axial tension, bending, and hydrostatic pressure

Method A ($\sigma_{a,Sd}$ is in tension)

The actual member axial stress, or the net axial stress, is estimated by subtracting the full capped-end axial compression from the calculated axial tension, $\sigma_{a,Sd}$. The net axial tensile stress, $(\sigma_{a,Sd} - \sigma_{q,Sd})$, is then used in Equation (6.34), which is a linear tension-bending interaction equation. There are mainly three effects due to the presence of external hydrostatic pressure: 1) a reduction of the axial tension due to the presence of a capped-end axial compression, 2) a reduction of the axial tensile strength, $f_{th,Rd}$, caused by the hoop compression, and 3) a reduction of the bending strength, $f_{mh,Rd}$, caused by the hoop compression.

As demonstrated in /3/, the axial tension-hydrostatic pressure interaction is similar to the bending-hydrostatic pressure interaction. The reduced axial tensile and bending strengths, as given by Equation (6.35) and Equation (6.36), were derived from the following ultimate-strength interaction equations:

Combined axial tension and hydrostatic pressure:

$$\left[\frac{\sigma_{a,Sd}}{f_d} \right]^2 + \left[\frac{\sigma_{p,Sd}}{f_{h,Rd}} \right]^{2\eta} + 2\nu \left[\frac{\sigma_{p,Sd}}{f_{h,Rd}} \right] \left[\frac{\sigma_{a,Sd}}{f_d} \right] = 1.0 \quad (12.3)$$

$$f_d = \frac{f_y}{\gamma_M}$$

$$\left[\frac{\sigma_{m,Sd}}{f_{m,Rd}} \right]^2 + \left[\frac{\sigma_{p,Sd}}{f_{h,Rd}} \right]^{2\eta} + 2\nu \left[\frac{\sigma_{p,Sd}}{f_{h,Rd}} \right] \left[\frac{\sigma_{m,Sd}}{f_{m,Rd}} \right] = 1.0 \quad (12.4)$$

$$f_{m,Rd} = \frac{f_m}{\gamma_M}$$

To obtain the axial tensile and bending strengths from the above two equations, the $\sigma_{a,Sd}$ and $\sigma_{m,Sd}$ terms are represented by f_{th} and f_{mh} , respectively, which are given by the positive roots of the quadratic equations.

When the calculated axial tensile stress is greater than or equal to the capped-end axial compression (i.e. $\sigma_{a,Sd} \geq \sigma_{q,Sd}$) the member is subjected to net axial tension. For this case, the member yield strength, f_y , is not replaced by a local buckling axial strength.

When the calculated axial tensile stress is less than the capped-end axial compression (i.e. $\sigma_{a,Sd} < \sigma_{q,Sd}$) the member is subjected to net axial compression and to a quasi-hydrostatic pressure condition. (A member is subjected to a pure hydrostatic pressure condition when the net axial compressive stress is equal to the capped-end axial stress, i.e. $\sigma_{a,Sd} = 0$.) Under this condition there is no member instability. Hence for this case, in which $\sigma_{a,Sd} \leq \sigma_{q,Sd}$, the cross-sectional yield criterion [Equation (6.39)] and the cross-sectional elastic buckling criterion [Equation (6.41)] need to be satisfied.

Method B ($\sigma_{ac,Sd}$ is in tension)

In this method the member net axial stress is the calculated value, $\sigma_{ac,Sd}$, since the effect of the capped-end axial compression is explicitly included in the design analysis. Therefore, the calculated axial tensile stress, $\sigma_{ac,Sd}$, can be used directly in the cross-sectional strength check, as given in Equation (6.42).

Comm. 6.3.9.2 Axial compression, bending, and hydrostatic pressure

Method A ($\sigma_{a,Sd}$ is in compression)

The capped-end axial compression due to hydrostatic pressure does not cause buckling of a member under combined external compression and hydrostatic pressure. The major contribution of the capped-end axial compression is earlier yielding of the member in the presence of residual stresses and additional external axial compression. The earlier yielding in turn results in a lower column buckling strength for the member, as given in Equation (6.47). When there is no hydrostatic pressure (i.e. $\sigma_{q,Sd} = 0$) Equation (6.47) reduces to the in-air case, see Equation (6.3).

It is incorrect to estimate the reduced column buckling strength by subtracting the capped-end axial compression from the in-air buckling strength calculated by Equation (6.3). This approach assumes that the capped-end axial compression can cause buckling and actually the reduced strength can be negative for cases where the capped-end axial compression is greater than the in-air buckling strength.

For the stability check [see Equation (6.43)], the calculated axial compression, $\sigma_{a,Sd}$, which is the additional external axial compression, is used. The effect of the capped-end axial compression is captured in the buckling strength, $f_{ch,Rd}$, which is derived for hydrostatic conditions. For strength or cross-sectional yield check [see Equation (6.44)], the net axial compression of the member is used. In addition, the cross-section elastic buckling criterion [see Equation (6.41)] need to be satisfied.

Method B ($\sigma_{ac,Sd}$ is in compression)

In this method the calculated axial stress, $\sigma_{ac,Sd}$, is the net axial compressive stress of the member since the capped-end axial compression is included in the design analysis. For the stability check [see Equation (6.50)], the axial compression to be used with the equation is the component that is in addition to the pure hydrostatic pressure condition. Therefore, the capped-end axial compression is subtracted from the net axial

compressive stress in Equation (6.50). For the strength check [see Eq. (6.51)], the net axial compressive stress is used.

When the calculated axial compressive stress is less than the capped-end axial compression, the member is under a quasi-hydrostatic pressure condition. That is, the net axial compression is less than the capped-end axial compression due to pure hydrostatic pressure. Under this loading, the member can not buckle as a beam-column. Of course, hoop collapse is still a limit state. For this case, in which $\sigma_{ac, Sd} \leq \sigma_{q, Sd}$, only the yield criterion of Equation (6.51) needs to be satisfied.

Comm 6.4 Tubular joints

Reasonable alternative methods to the requirements in this standard may be used for the design of joints. Test data and analytical techniques may be used as a basis for design, provided that it is demonstrated that the resistance of such joints can be reliably estimated. The recommendations presented here have been derived from a consideration of the characteristic strength of tubular joints. Characteristic strength is comparable to lower bound strength. Care should therefore be taken in using the results of limited tests programs or analytical investigations to provide an estimate of joint resistance. Consideration shall be given to the imposition of a reduction factor on the calculation of joint resistance to account for a small amount of data or a poor basis for the calculation. Analytical or numerical techniques should be calibrated and benchmarked to suitable test data.

The formulas in this subclause are based upon-API RP-2A /1/ and paper by Dier et al. /16/.

Comm. 6.4.1 General

Detailing practice

Joint detailing is an essential element of joint design. For simple tubular joints, the recommended detailing nomenclature and dimensioning are shown in Figure 6-1. This practice indicates that if an increased wall thickness of chord or special steel, is required, it should extend past the outside edge of incoming bracing a minimum of one quarter of the chord diameter or 300 mm, whichever is greater. Short chord can lengths can lead to a downgrading of joint resistance. The designer should consider specifying an increase of such chord can length to remove the need for resistance downgrading, see 6.4.3.5. An increased wall thickness of brace or special steel, if required, should extend a minimum of one brace diameter or 600 mm, whichever is greater. Neither the cited chord can nor brace stub dimension includes the length over which thickness taper occurs.

The minimum nominal gap between adjacent braces, whether in- or out-of-plane, is normally 50 mm. Care should be taken to ensure that overlap of welds at the toes of the joint is avoided. When overlapping braces occur, the amount of overlap should preferably be at least $d/4$ (where d is diameter of the through brace) or 150 mm, whichever is greater. This dimension is measured along the axis of the through member.

Where overlapping of braces is necessary or preferred, the brace with the larger wall thickness should be the through brace and fully welded to the chord. Further, where substantial overlap occurs, the larger diameter brace should be specified as the through member. This brace require an end stub to ensure that the thickness is at least equal to that of the overlapping brace.

Comm. 6.4.2 Joint classification

Case (h) in Figure 6-2 is a good example of the actions and classification hierarchy that should be adopted in the classification of joints. Replacement of brace actions by a combination of tension and compression force to give the same net action is not permitted. For example, replacing the force in the horizontal brace on the left hand side of the joint by a compression force of 1000 and tension force of 500 is not permitted, as this may result in an inappropriate X classification for this horizontal brace and a K classification for the diagonal brace.

Special consideration should be given to establish the proper gap if a portion of the action is related to K-joint behaviour. The most obvious case in Figure 6-2 is (a), for which the appropriate gap is between adjacent braces. However, if an intermediate brace exists, as in case (d), the appropriate gap is between the outer loaded braces. In this case, since the gap is often large, the K-joint resistance could revert to that of a Y-joint. Case (e) is instructive in that the appropriate gap for the middle brace is gap 1, whereas for the top brace it is gap 2. Although the bottom brace is treated as 100 % K classification, a weighted average in

resistance is required, depending on how much of the acting axial force in this brace is balanced by the middle brace (gap 1) and how much is balanced by the top brace (gap 2).

Comm. 6.4.3.3 Strength factor Q_u

The Q_u term for tension forces is based on limiting the resistance to first crack.

Comm. 6.4.3.4 Chord action factor Q_f

The Q_f factor is similar as in API RP-2A /1/ for brace in compression for X-joints. For X joints with the brace in tension modified values for the Q_f factor are recommended. The API formulas are based on test and analyses only with the brace in compression (see /17/ and /18/) and it is judged that the influence of chord stress on the joint strength will be different when the brace is in tension. As test and analytical results are not available the recommended Q_f factors in the case of X-joints with the brace in tension are established by the following assumptions: For $\beta = 1.0$ the Q_f factor for the case of brace in tension is assumed to be mirrored from the compression case. For $\beta < 0.9$ it is assumed that when the brace is in tension and the chord stress in compression the Q_f factor to be close to the case for the brace in compression with tension chord stress. (Mirrored) For the case with the brace in tension and the chord stress in tension the Q_f factor is assumed as mirror symmetry of the case with the chord stress in compression. See Figure 12-6.

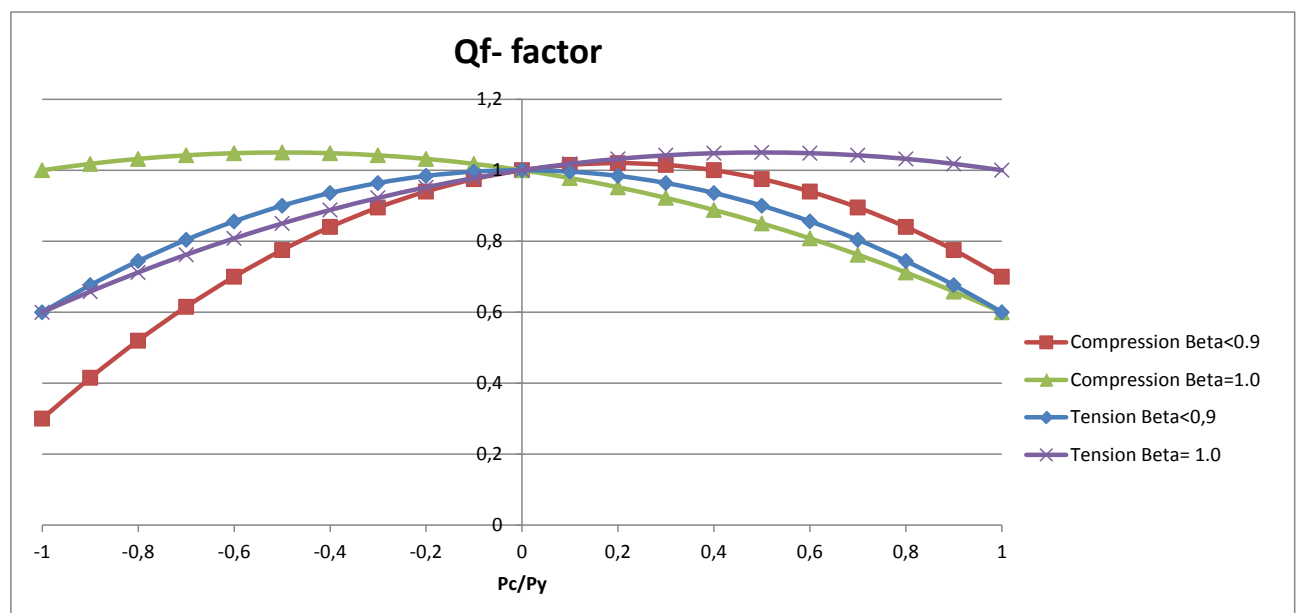


Figure 12-6 Q_f factor as a function of chord load for brace tension and compression for two β ratios

Comm. 6.4.5 Ringstiffened joints

For ring-stiffened joints, the load effects determined by elastic theory will, in general, include local stress peaks. In the ULS check, such peaks may be reduced to mean values within limited areas. The extent of these areas shall be evaluated for the actual geometry. An assumed redistribution of stress should not lead to significant change in the equilibrium of the different parts of the joint. For example, if an action in a brace is resisted by a shear force over a ring-stiffener with an associated moment, a removal of local stress peaks should not imply significant reduction in this moment required for equilibrium.

Ring-stiffened joints may be designed according to plastic theory provided that all parts of the joint belong to cross section class 1 or 2 (see EN-1993-1-1 for definitions). Load effects may be determined by assuming relevant plastic collapse mechanisms. The characteristic resistance shall be determined by recognised methods of plastic theory. The design resistance is determined by dividing the characteristic resistance by $\gamma_M = 1.15$.

The resistance may also be determined based on non-linear analysis. The computer programme used for such analysis should be validated as providing reliable results in comparison with other analysis and tests.

Also the type of input to the analysis should be calibrated to provide reliable results. This includes type of element used, element mesh and material description in terms of stress strain relationship.

Comm. 6.5 Strength of conical transitions

The formulas are based on ISO 19902, but the influence of local bending stress is changed in revision 3. Similar to what is assumed in API RP-2A /1/ it is assumed that the ultimate strength limit need not be limited to yield in extreme fibre when the stresses from local bending is included. In reality these stresses are not necessary for the connection to be in equilibrium with the external loads when ultimate strength is considered. It is uncertain how important the local bending stresses are for the ultimate capacity of the connection, but is assumed to be well to the conservative side to include the local bending moment according to plastic capacity of the shell wall. It is furthermore assumed that the local bending stresses are not detrimental to the buckling capacity of the tubular or the cone sections.

Comm. 6.5.3.1 Local buckling under axial compression

Platforms generally have a very small number of cones. Thus, it might be more expeditious to design the cones with a geometry such that the axial resistance is equal to that of yield, see 6.3.3.

Comm. 6.5.3.2 Junction yielding

The resistance of the junction is checked according to von Mises yield criterion when the hoop stress is tensile.

Comm. 6.5.4.1 Hoop buckling

Hoop buckling is analysed similarly to that of a tubular subjected to external pressure using equivalent geometry properties.

Comm. 6.5.4.2 Junction yielding and buckling

The load effect from external pressure is directly added to the existing stress at the junction for utilisation check with respect to yielding and buckling.

Comm. 6.5.5.2 Junction rings without external hydrostatic pressure

The resistance of stiffeners at a junction is checked as a ring where the effective area of the tubular and the cone is added to that of the ring section.

Comm. 6.5.5.3 Junction rings with external hydrostatic pressure

The required moment of inertia is derived as the sum of that required for the junction itself and that due to external pressure.

Comm. 6.5.5.4 Intermediate stiffening rings

Design of intermediate ring stiffeners within a cone is performed along the same principles as used for design of ring stiffeners in tubulars.

Comm. 6.7 Design of cylindrical shells

A tubular section in air with diameter/thickness ratio larger than 60 is likely to fail by local buckling at an axial stress less than the material yield strength. Based on NS-ENV 1993 1-1 the upper limit of section class 3, where an axial stress equal yield strength can be achieved, is a D/t ratio of $21150/f_y$ where f_y is material yield strength in MPa. The resistance of members failing due to local buckling is more sensitive to geometric imperfections than members that can sustain yielding over the thickness and allow some redistribution of local stresses due to yielding. The failure of such members is normally associated with a descending post-critical behaviour that may be compared more with that of a brittle structure, i. e. the redistribution of load cannot be expected. Structures with this behaviour are denoted as shells. A definition of a shell structure should not only include geometry and material resistance, but also loading as the axial resistance is reduced

by e. g. increasing pressure. Design equations have been developed to account for different loading conditions, see /14/. The background for these design equations is given by /2/.

Comm. 9 Accidental damage limit states

Examples of failure criteria are as follows:

- Critical deformation criteria defined by integrity of passive fire protection.
To be considered for walls resisting explosion pressure and shall serve as fire barrier after the explosion.
- Critical deflection for structures to avoid damage to process equipment (riser, gas pipe, etc). To be considered for structures or part of structures exposed to impact loads as ship collision, dropped object etc.
- Critical deformation to avoid leakage of compartments. To be considered in case of impact against floating structures where the acceptable collision damage is defined by the minimum number of undamaged compartments to remain stable.

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DESIGN OF STEEL STRUCTURES
ANNEX A
DESIGN AGAINST ACCIDENTAL ACTIONS

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A.1 SYMBOLS

A	Cross-sectional area
A_e	Effective area of stiffener and effective plate flange
A_s	Area of stiffener
A_p	Projected cross-sectional area
A_w	Shear area of stiffener/girder
B	Width of contact area
C_D	Hydrodynamic drag coefficient
D	Diameter of circular sections, plate stiffness
E	Young's Modulus of elasticity, $2.1 \cdot 10^5$
E_p	Plastic modulus
E_{kin}	Kinetic energy
E_s	Strain energy
F	Lateral load, total load
A_w	Shear area of stiffener/girder
H	Non-dimensional plastic stiffness
I	Moment of inertia, impuls
J	Mass moment of inertia
K_l	Load transformation factor
K_m	Mass transformation factor
K_{lm}	Load-mass transformation factor
L	Beam length
M	Total mass, cross-sectional moment
M_p	Plastic bending moment resistance
N_p	Plastic axial resistance
T	Fundamental period of vibration
N	Axial force
N_{sd}	Design axial compressive force
N_{Rd}	Design axial compressive capacity
N_p	Axial resistance of cross section
R	Resistance
R_0	Plastic collapse resistance in bending
V	Volume, displacement
W_p	Plastic section modulus
W	Elastic section modulus
a	Added mass
a_s	Added mass for ship
a_i	Added mass for installation

b	Width of collision contact zone
b_f	Flange width
c	Factor
c_f	Axial flexibility factor
c_{lp}	Plastic zone length factor
c_s	Shear factor for vibration eigenperiod
c_Q	Shear stiffness factor
c_w	Displacement factor for strain calculation
d	Smaller diameter of threaded end of drill collar
d_c	Characteristic dimension for strain calculation
$\bar{f}$	Generalised load
f_y	Characteristic yield strength
f_u	Ultimate material tensile strength
g	Acceleration of gravity, 9.81 m/s^2
h_w	Web height for stiffener/girder
i	Radius of gyration
$\bar{k}$	Generalised stiffness
k	Stiffness, equivalent stiffness, plate stiffness
k_1	Bending stiffness in linear domain for beam
k_1'	Stiffness in linear domain including shear deformation
k_Q	Shear stiffness in linear domain for beam
$k_{y,\theta}$	Temperature reduction of effective yield stress for maximum temperature in connection
l	Plate length, beam length
m	Distributed mass
m_s	Ship mass
m_i	Installation mass
m_{eq}	Equivalent mass
$\bar{m}$	Generalised mass
p	Explosion pressure
r	Radius of deformed area
r_c	Plastic collapse resistance in bending for plate
s	Distance, stiffener spacing
s_c	Characteristic distance
s_e	Effective width of plate
t	Thickness, time
t_d	Duration of explosion
t_f	Flange thickness
t_w	Web thickness
v_s	Velocity of ship
v_i	Velocity of installation
v_t	Terminal velocity

w	Deformation, displacement
w_c	Characteristic deformation
w_d	dent depth
$\overline{w}$	Non-dimensional deformation
x	Axial coordinate
y	Generalised displacement, displacement amplitude
y_{el}	Generalised displacement at elastic limit
z	Distance from pivot point to collision point
α	Plate aspect parameter
β	Cross-sectional slenderness factor
ε	Yield strength factor, strain
ε_{cr}	Critical strain for rupture
ε_y	Yield strain
η	Plate eigenperiod parameter
ϕ	Displacement shape function
$\overline{\lambda}$	Reduced slenderness ratio
μ	Ductility ratio
ν	Poisson's ratio, 0.3
θ	Angle
ρ	Density of steel, 7860 kg/m ³
ρ_w	Density of sea water, 1025 kg/m ³
τ	Shear stress
τ_{cr}	Critical shear stress for plate plugging
ξ	Interpolation factor
ψ	Plate stiffness parameter

A.2 GENERAL

This Annex deals with the design to maintain the load-bearing function of the structures during accidental events. The overall goal of the design against accidental actions is to achieve a system where the main safety functions of the installation are not impaired.

Design Accidental Actions and associated performance criteria are determined by Quantified Risk Assessment (QRA), see NORSOK N-003 /1/.

In conjunction with design against accidental actions, performance criteria may need to be formulated such that the structure or components or sub-assemblies thereof - during the accident or within a certain time period after the accident - shall not impair the main safety functions such as:

- usability of escapeways,
- integrity of shelter areas,
- global load bearing capacity.

The performance criteria derived will typically be related to:

- energy dissipation
- local strength
- resistance to deformation (e.g braces in contact with risers/caissons, use of escape ways)
- endurance of fire protection
- ductility (allowable strains) - to avoid cracks in components, fire walls, passive fire protection etc.

The inherent uncertainty of the frequency and magnitude of the accidental loads as well as the approximate nature of the methods for determination analysis of accidental load effects shall be recognised. It is therefore essential to apply sound engineering judgement and pragmatic evaluations in the design.

The material factor to be used for checks of accidental limit states is $\gamma_M = 1.0$ unless noted otherwise. If determination of resistance is made according to NS-EN 1993-1-1, NS-EN 1993-1-5 or NS-EN 1993-1-8, the material factor should be determined as given in subclause 9.1.

A.3 SHIP COLLISIONS

A.3.1 General

The ship collision action is characterised by a kinetic energy, governed by the mass of the ship, including hydrodynamic added mass and the speed of the ship at the instant of impact. Depending upon the impact conditions, a part of the kinetic energy may remain as kinetic energy after the impact. The remainder of the kinetic energy has to be dissipated as strain energy in the installation and, possibly, in the vessel. Generally this involves large plastic strains and significant structural damage to either the installation or the ship or both. The strain energy dissipation is estimated from force-deformation relationships for the installation and the ship, where the deformations in the installation shall comply with ductility and stability requirements.

The load bearing function of the installation shall remain intact with the damages imposed by the ship collision action. In addition, the residual strength requirements given in Section A.7 shall be complied with.

The structural effects from ship collision may either be determined by non-linear dynamic finite element analyses or by energy considerations combined with simple elastic-plastic methods.

If non-linear dynamic finite element analysis is applied all effects described in the following paragraphs shall either be implicitly covered by the modelling adopted or subjected to special considerations, whenever relevant.

Often the integrity of the installation can be verified by means of simple calculation models.

If simple calculation models are used the part of the collision energy that needs to be dissipated as strain energy can be calculated by means of the principles of conservation of momentum and conservation of energy, refer Section A.3.3.

It is convenient to consider the strain energy dissipation in the installation to take part on three different levels:

- local cross-section
- component/sub-structure
- total system

Interaction between the three levels of energy dissipation shall be considered.

Plastic modes of energy dissipation shall be considered for cross-sections and component/substructures in direct contact with the ship. Elastic strain energy can in most cases be disregarded, but elastic axial flexibility may have a substantial effect on the load-deformation relationships for components/sub-structures. Elastic energy may contribute significantly on a global level.

A.3.2 Design principles

With respect to the distribution of strain energy dissipation there may be distinguished between, see Figure A.3-1:

- strength design
- ductility design
- shared-energy design

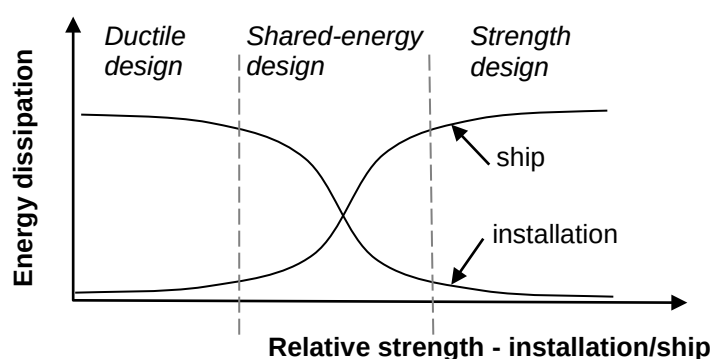


Figure A.3-1 Energy dissipation for strength, ductile and shared-energy design

Strength design implies that the installation is strong enough to resist the collision force with minor deformation, so that the ship is forced to deform and dissipate the major part of the energy.

Ductility design implies that the installation undergoes large, plastic deformations and dissipates the major part of the collision energy.

Shared energy design implies that both the installation and ship contribute significantly to the energy dissipation.

From calculation point of view strength design or ductility design is favourable. In this case the response of the «soft» structure can be calculated on the basis of simple considerations of the geometry of the «rigid» structure. In shared energy design both the magnitude and distribution of the collision force depends upon the deformation of both structures. This interaction makes the analysis more complex.

In most cases ductility or shared energy design is used. However, strength design may in some cases be achievable with little increase in steel weight.

A.3.3 Collision mechanics

A.3.3.1 Strain energy dissipation

The collision energy to be dissipated as strain energy may - depending on the type of installation and the purpose of the analysis - be taken as:

Compliant installations

$$E_s = \frac{1}{2}(m_s + a_s)v_s^2 \frac{\left(1 - \frac{v_i}{v_s}\right)^2}{1 + \frac{m_s + a_s}{m_i + a_i}} \quad (\text{A.3.1})$$

Fixed installations

$$E_s = \frac{1}{2}(m_s + a_s)v_s^2 \quad (\text{A.3.2})$$

Articulated columns

$$E_s = \frac{1}{2}(m_s + a_s) \frac{\left(1 - \frac{v_i}{v_s}\right)^2}{1 + \frac{m_s z^2}{J}} \quad (\text{A.3.3})$$

m_s = ship mass
 a_s = ship added mass

v_s	=	impact speed
m_i	=	mass of installation
a_i	=	added mass of installation
v_i	=	velocity of installation
J	=	mass moment of inertia of installation (including added mass) with respect to effective pivot point
z	=	distance from pivot point to point of contact

In most cases the velocity of the installation can be disregarded, i.e. $v_i = 0$.

The installation can be assumed compliant if the duration of impact is small compared to the fundamental period of vibration of the installation. If the duration of impact is comparatively long, the installation can be assumed fixed.

Jacket structures can normally be considered as fixed. Floating platforms (semi-submersibles, TLP's, production vessels) can normally be considered as compliant. Jack-ups may be classified as fixed or compliant.

A.3.3.2 Reaction force to deck

In the acceleration phase the inertia of the topside structure generates large reaction forces. An upper bound of the maximum force between the collision zone and the deck for bottom supported installations may be obtained by considering the platform compliant for the assessment of total strain energy dissipation and assume the platform fixed at deck level when the collision response is evaluated.

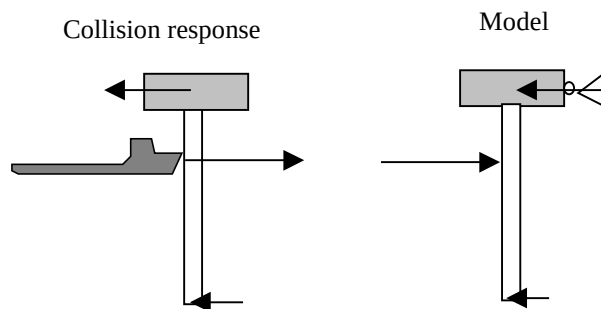


Figure A.3-2 Model for assessment of reaction force to deck

A.3.4 Dissipation of strain energy

The structural response of the ship and installation can formally be represented as load-deformation relationships as illustrated in Figure A.3-3. The strain energy dissipated by the ship and installation equals the total area under the load-deformation curves.

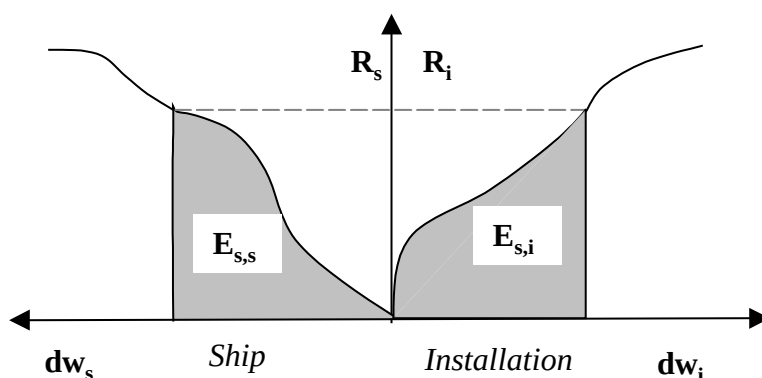


Figure A.3-3 Dissipation of strain energy in ship and platform

$$E_s = E_{s,s} + E_{s,i} = \int_0^{w_{s,max}} R_s dw_s + \int_0^{w_{i,max}} R_i dw_i \quad (\text{A.3.4})$$

As the load level is not known a priori an incremental procedure is generally needed.

The load-deformation relationships for the ship and the installation are often established independently of each other assuming the other object infinitely rigid. This method may have, however, severe limitations; both structures will dissipate some energy regardless of the relative strength.

Often the stronger of the ship and platform will experience less damage and the softer more damage than what is predicted with the approach described above. As the softer structure deforms the impact force is distributed over a larger contact area. Accordingly, the resistance of the strong structure increases. This may be interpreted as an "upward" shift of the resistance curve for the stronger structure (refer Figure A.3-3).

Care should be exercised that the load-deformation curves calculated are representative for the true, interactive nature of the contact between the two structures.

A.3.5 Ship collision forces

A.3.5.1 Recommended force-deformation relationships

Force-deformation relationships for a supply vessels with a displacement of 5000 tons are given in Figure A.3-4 for broad side -, bow-, stern end and stern corner impact for a vessel with stern roller.

The curves for broad side and stern end impacts are based upon penetration of an infinitely rigid, vertical cylinder with a given diameter and may be used for impacts against jacket legs ($D = 1.5 \text{ m}$) and large diameter columns ($D = 10 \text{ m}$).

The curve for stern corner impact is based upon penetration of an infinitely rigid cylinder and may be used for large diameter column impacts.

In lieu of more accurate calculations the curves in Figure A.3-4 may be used for square-rounded columns.

For supply vessels and merchant vessels in the range of 2-5000 tons displacement, the force deformation relationships given in Figure A.3-5 may be used for impacts against jacket legs with diameter 1.5 m – 2.5 m. The force deformation relationships given are for conventional supply ship without e.g. bow reinforcements for operations in ice.

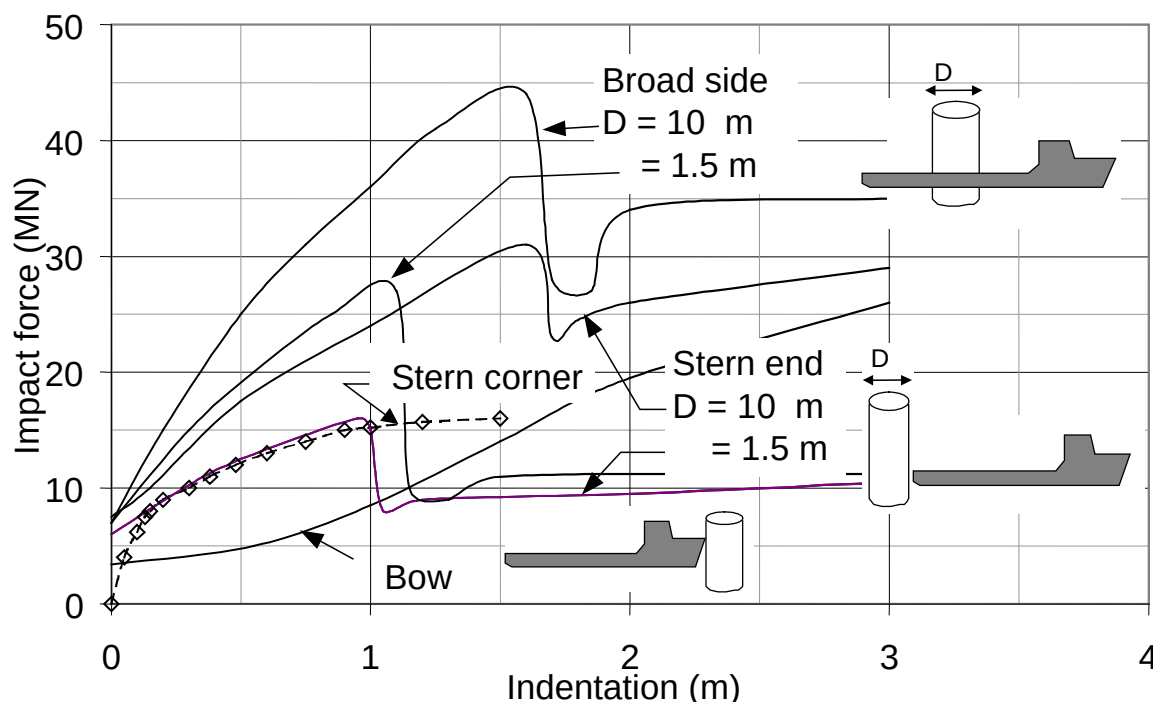


Figure A.3-4 Recommended-deformation curve for beam, bow and stern impact

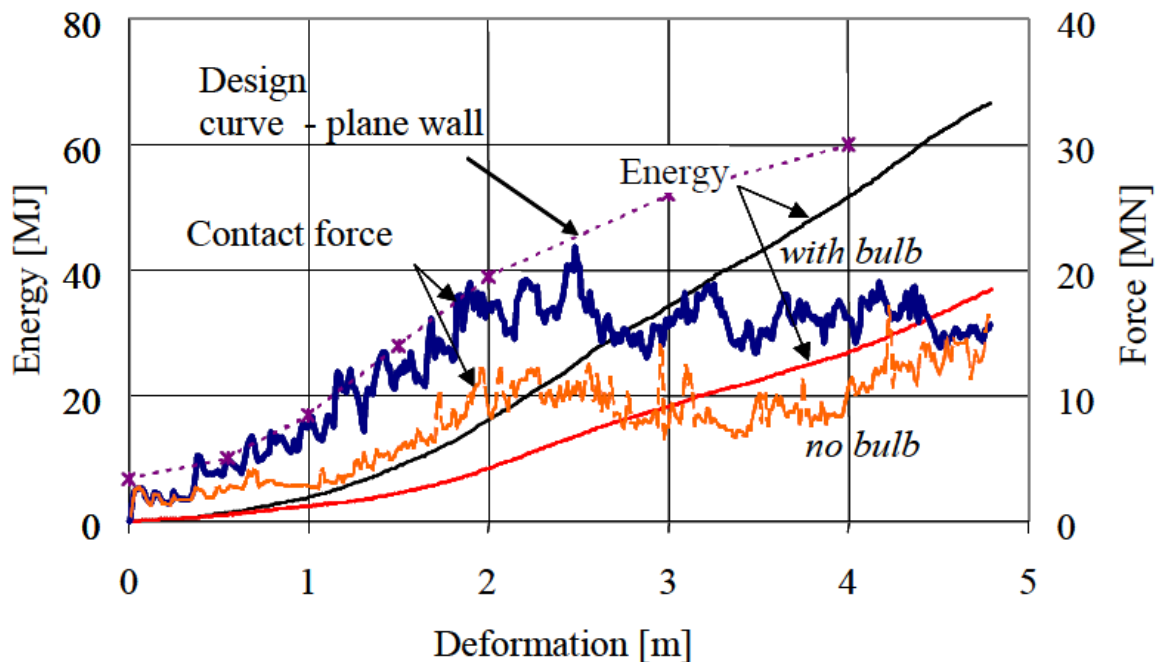


Figure A.3-5 Force -deformation relationship for bow with and without bulb (2-5.000 dwt)

The curve for bow impact is based upon collision with an infinitely rigid, plane wall and may be used for large diameter column impacts, but should not be used for significantly different collision events, e.g. impact against tubular braces.

For beam -, stern end – and stern corner impacts against jacket braces all energy shall normally be assumed dissipated by the brace, refer Comm. A.3.5.2.

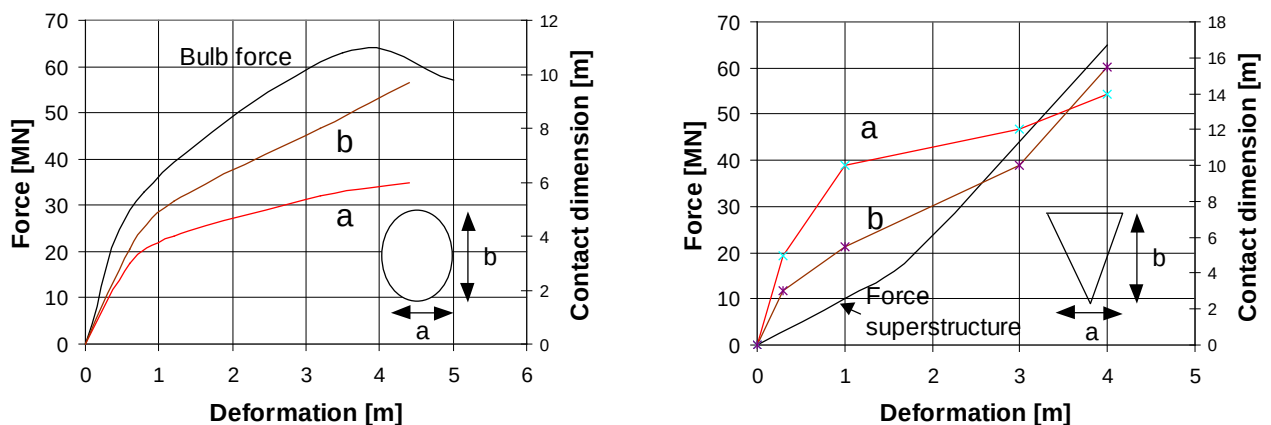


Figure A.3-6 Force -deformation relationship for tanker bow impact (~ 125.000 dwt)

Force-deformation relationships for tanker bow impact is given in Figure A.3-6 for the bulbous part and the superstructure, respectively. The curves may be used provided that the impacted structure (e.g. stern of floating production vessels) does not undergo substantial deformation i.e. strength design requirements are complied with. If this condition is not met interaction between the bow and the impacted structure shall be taken into consideration. Non-linear finite element methods or simplified plastic analysis techniques of members subjected to axial crushing shall be employed [3/, /5/.

A.3.5.2 Force contact area for strength design of large diameter columns.

The basis for the curves in Figure A.3-4 is strength design, i.e. limited local deformations of the installation at the point of contact. In addition to resisting the total collision force, large diameter columns have to resist

local concentrations (subsets) of the collision force, given for stern corner impact in Table A.3-1 and stern end impact in Table A.3-2.

Table A.3-1 Local concentrated collision force -evenly distributed over a rectangular area. Stern corner impact

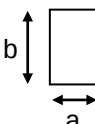
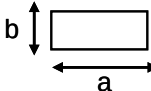
Contact area		Force (MN)	
a(m)	b (m)		
0.35	0.65	3.0	
0.35	1.65	6.4	
0.20	1.15	5.4	

Table A.3-2 Local concentrated collision force -evenly distributed over a rectangular area. Stern end impact

Contact area		Force (MN)	
a (m)	b(m)		
0.6	0.3	5.6	
0.9	0.5	7.5	
2.0	1.1	10	

If strength design is not aimed for - and in lieu of more accurate assessment (e.g. nonlinear finite element analysis) - all strain energy has to be assumed dissipated by the column, corresponding to indentation by an infinitely rigid stern corner.

A.3.5.3 Energy dissipation in ship bow

For typical supply vessels bows and bows of merchant vessels of similar size (i.e. 2-5000 tons displacement), energy dissipation in the ship bow may be taken into account provided that the collapse resistance in bending for the brace, R_0 , see Section A.3.7, is according to the values given in Table A.3-3. The figures are valid for normal bows without ice strengthening and for brace diameters less than 1.25 m. The values should be used as step functions, i.e. interpolation for intermediate resistance levels is not allowed. If contact location is not governed by operation conditions, size of ship and platform etc., the values for arbitrary contact location shall be used. (see also Comm. A.3.5.3)

Table A.3-3 Energy dissipation in bow versus brace resistance

Contact location	Energy dissipation in bow if brace resistance R_0			
	> 3 MN	> 6 MN	> 8 MN	> 10 MN
Above bulb	1 MJ	4 MJ	7 MJ	11 MJ
First deck	0 MJ	2 MJ	4 MJ	17 MJ
First deck - oblique brace	0 MJ	2 MJ	4 MJ	17 MJ
Between f'castle/first deck	1 MJ	5 MJ	10 MJ	15 MJ
Arbitrary loaction	0 MJ	2 MJ	4 MJ	11 MJ

In addition , the brace cross-section must satisfy the following compactness requirement

$$f_y t^{1.5} D^{0.5} \geq \frac{2}{3} \cdot \text{factor} \quad (\text{A.3.5})$$

where factor is the required resistance in [MN] given in Table A.3-3.

See Section A.3.6 for notation.

If the brace is designed to comply with these provisions, special care should be exercised that the joints and adjacent structure is strong enough to support the reactions from the brace.

A.3.6 Force-deformation relationships for denting of tubular members

The contribution from local denting to energy dissipation is small for brace members in typical jackets and should be neglected.

The resistance to indentation of unstiffened tubes may be taken from Figure A.3-7. Alternatively, the resistance may be calculated from Equation (A.3.6):

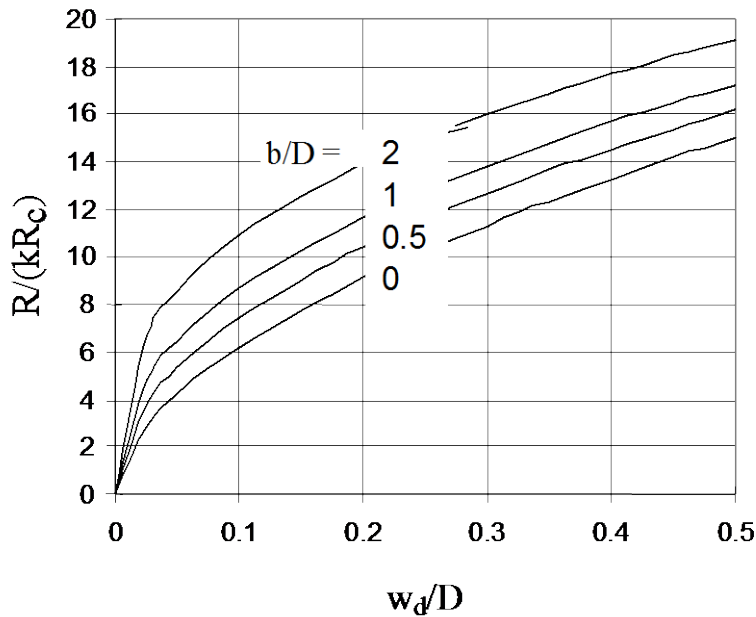


Figure A.3-7 Resistance curve for local denting

$$\frac{R}{R_c} = k c_1 \left(\frac{w_d}{D} \right)^{c_2} \quad (\text{A.3.6})$$

$$R_c = f_y \frac{t^2}{4} \sqrt{\frac{D}{t}}$$

$$c_1 = 22 + 1.2 \frac{B}{D}$$

$$c_2 = \frac{1.925}{3.5 + \frac{B}{D}}$$

$$k = 1.0 \quad \frac{N_{Sd}}{N_{Rd}} \leq 0.2$$

$$k = 1.0 - 2 \left(\frac{N_{Sd}}{N_{Rd}} - 0.2 \right) \quad 0.2 < \frac{N_{Sd}}{N_{Rd}} < 0.6$$

$$k = 0 \quad 0.6 \leq \frac{N_{Sd}}{N_{Rd}}$$

N_{Sd}	=	design axial compressive force
N_{Rd}	=	design axial compressive resistance
B	=	width of contact area
w_d	=	dent depth

The curves are inaccurate for small indentation, and they should not be used to verify a design where the dent damage is required to be less than $\frac{w_d}{D} < 0.05$

A.3.7 Force-deformation relationships for beams

A.3.7.1 General

The response of a beam subjected to a collision load is initially governed by bending, which is affected by and interacts with local denting under the load. The bending capacity is also reduced if local buckling takes place on the compression side. As the beam undergoes finite deformations, the load carrying capacity may increase considerably due to the development of membrane tension forces. This depends upon the ability of adjacent structure to restrain the connections at the member ends to inward displacements. Provided that the connections do not fail, the energy dissipation capacity is either limited by tension failure of the member or rupture of the connection.

Simple plastic methods of analysis are generally applicable. Special considerations shall be given to the effect of :

- elastic flexibility of member/adjacent structure
- local deformation of cross-section
- local buckling
- strength of connections
- strength of adjacent structure
- fracture

A.3.7.2 Plastic force-deformation relationships including elastic, axial flexibility

Relatively small axial displacements have a significant influence on the development of tensile forces in members undergoing large lateral deformations. An equivalent elastic, axial stiffness may be defined as

$$\frac{1}{K} = \frac{1}{K_{\text{node}}} + \frac{\ell}{2EA} \quad (\text{A.3.7})$$

K_{node} = axial stiffness of the node with the considered member removed. This may be determined by introducing unit loads in member axis direction at the end nodes with the member removed.

Plastic force-deformation relationship for a central collision (midway between nodes) may be obtained from :

- Figure A.3-8 for tubular members
- Figure A.3-9 for stiffened plates in lieu of more accurate analysis.

The following notation applies:

$R_0 = \frac{4c_1 M_p}{\ell}$	plastic collapse resistance in bending for the member, for the case that contact point is at mid span
$\bar{w} = \frac{w}{c_1 w_c}$	non-dimensional deformation
$c = \frac{4c_1 K w_c^2}{f_y A \ell}$	non-dimensional spring stiffness
$c_1 = 2$	for clamped beams
$c_1 = 1$	for pinned beams

$$w_c = \frac{D}{2} \quad \text{characteristic deformation for tubular beams}$$

$$w_c = \frac{1.2W_p}{A} \quad \text{characteristic deformation for stiffened plating}$$

$$W_p = \quad \text{plastic section modulus}$$

$$\ell = \quad \text{member length}$$

For non-central collisions the force-deformation relationship may be taken as the mean value of the force-deformation curves for central collision with member half-length equal to the smaller and the larger portion of the member length, respectively.

For members where the plastic moment capacity of adjacent members is smaller than the moment capacity of the impacted member the force-deformation relationship may be interpolated from the curves for pinned ends and clamped ends:

$$R = \xi R_{\text{clamped}} + (1 - \xi) R_{\text{pinned}} \quad (\text{A.3.8})$$

where

$$0 \leq \xi = \frac{R_0^{\text{actual}}}{4 \frac{M_p}{\ell}} - 1 \leq 1 \quad (\text{A.3.9})$$

$$R_0^{\text{actual}} = \quad \text{Plastic resistance by bending action of beam accounting for actual bending resistance of adjacent members}$$

$$R_0^{\text{actual}} = \frac{4M_p + 2M_{p1} + 2M_{p2}}{\ell} \quad (\text{A.3.10})$$

$$M_{pj} = \sum_i M_{pj,i} \leq M_p \quad i = \text{adjacent member no } i, \quad j = \text{end number } \{1,2\} \quad (\text{A.3.11})$$

$M_{pj,i}$ = Plastic bending resistance for member no. i.

Elastic, rotational flexibility of the node is normally of moderate significance

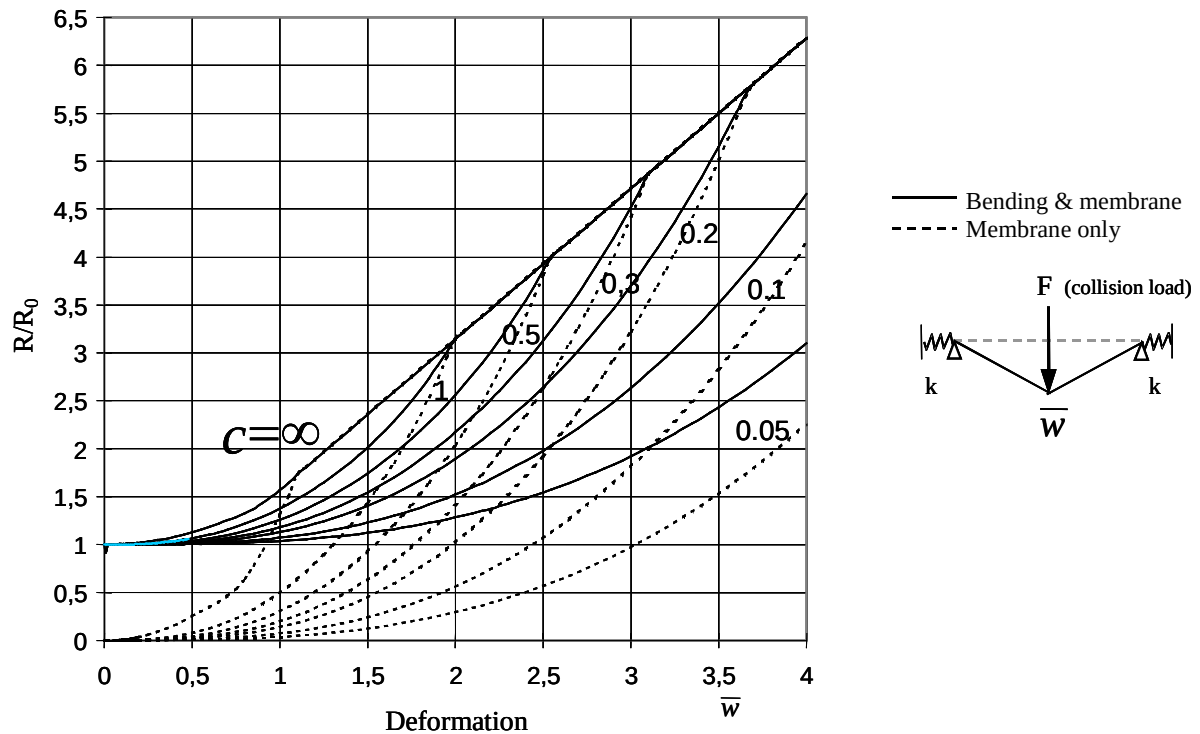


Figure A.3-8 Force-deformation relationship for tubular beam with axial flexibility

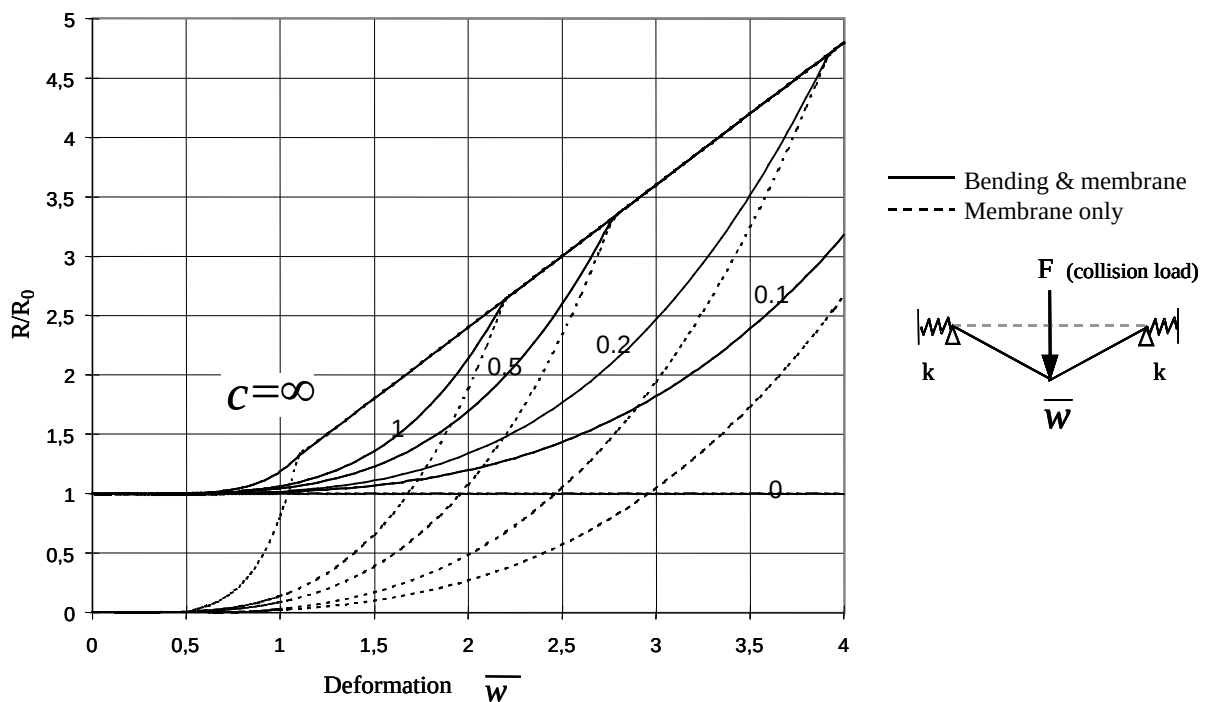


Figure A.3-9 Force-deformation relationship for stiffened plate with axial flexibility.

A.3.7.3 Bending capacity of dented tubular members

The reduction in plastic moment capacity due to local denting shall be considered for members in compression or moderate tension, but can be neglected for members entering the fully plastic membrane state.

Conservatively, the flat part of the dented section according to the model shown in Figure A.3-10 may be assumed non-effective. This gives:

$$\frac{M_{red}}{M_p} = \cos \frac{\theta}{2} - \frac{1}{2} \sin \theta \quad (\text{A.3.12})$$

$$M_p = f_y D^2 t$$

$$\theta = \arccos \left(1 - \frac{2w_d}{D} \right)$$

w_d = dent depth as defined in Figure A.3-10.

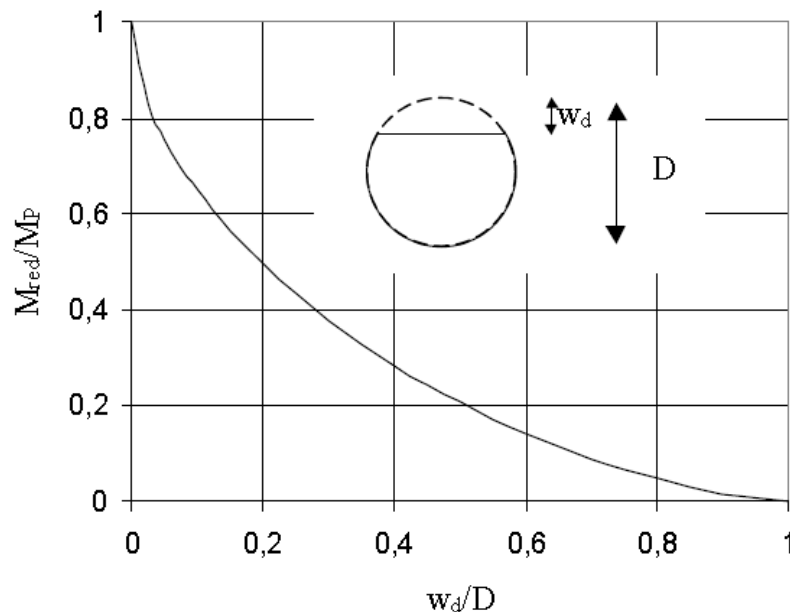


Figure A.3-10 Reduction of moment capacity due to local dent

A.3.8 Strength of connections

Provided that large plastic strains can develop in the impacted member, the strength of the connections that the member frames into has to be checked.

The resistance of connections should be taken from ULS requirements in this standard for tubular joints and Eurocode 3 or NS3472 for other joints.

For braces reaching the fully plastic tension state, the connection shall be checked for a load equal to the axial resistance of the member. The design axial stress shall be assumed equal to the ultimate tensile strength of the material.

If the axial force in a tension member becomes equal to the axial capacity of the connection, the connection has to undergo gross deformations. The energy dissipation will be limited and rupture has to be considered at a given deformation. A safe approach is to assume disconnection of the member once the axial force in the member reaches the axial capacity of the connection.

If the capacity of the connection is exceeded in compression and bending, this does not necessarily mean failure of the member. The post-collapse strength of the connection may be taken into account provided that such information is available.

A.3.9 Strength of adjacent structure

The strength of structural members adjacent to the impacted member/sub-structure must be checked to see whether they can provide the support required by the assumed collapse mechanism. If the adjacent structure fails, the collapse mechanism must be modified accordingly. Since, the physical behaviour becomes more complex with mechanisms consisting of an increasing number of members it is recommended to consider a design which involves as few members as possible for each collision scenario.

A.3.10 Ductility limits

A.3.10.1 General

The maximum energy that the impacted member can dissipate will – ultimately - be limited by local buckling on the compressive side or fracture on the tensile side of cross-sections undergoing finite rotation.

If the member is restrained against inward axial displacement, any local buckling must take place before the tensile strain due to membrane elongation overrides the effect of rotation induced compressive strain.

If local buckling does not take place, fracture is assumed to occur when the tensile strain due to the combined effect of rotation and membrane elongation exceeds a critical value.

To ensure that members with small axial restraint maintain moment capacity during significant plastic rotation it is recommended that cross-sections be proportioned to Class 1 requirements, defined in Eurocode 3 (EN-1993-1-1).

Initiation of local buckling does, however, not necessarily imply that the capacity with respect to energy dissipation is exhausted, particularly for Class 1 and Class 2 cross-sections. The degradation of the cross-sectional resistance in the post-buckling range may be taken into account provided that such information is available, refer Comm. A.3.10.1

For members undergoing membrane stretching a lower bound to the post-buckling load-carrying capacity may be obtained by using the load-deformation curve for pure membrane action.

A.3.10.2 Local buckling

Circular cross-sections:

Buckling does not need to be considered for a beam with axial restraints if the following condition is fulfilled:

$$\beta \leq \left(\frac{14c_f f_y}{c_1} \left(\frac{\kappa \ell}{d_c} \right)^2 \right)^{\frac{1}{3}} \quad (\text{A.3.13})$$

where

$$\beta = \frac{D/t}{235/f_y} \quad (\text{A.3.14})$$

axial flexibility factor

$$c_f = \left(\frac{\sqrt{c}}{1 + \sqrt{c}} \right)^2 \quad (\text{A.3.15})$$

d_c	=	characteristic dimension
	=	D for circular cross-sections
c_1	=	2 for clamped ends
	=	1 for pinned ends
c	=	non-dimensional spring stiffness, refer Section A.3.7.2
$\kappa \ell \leq 0.5 \ell$	=	the smaller distance from location of collision load to adjacent joint

If this condition is not met, buckling may be assumed to occur when the lateral deformation exceeds

$$\frac{w}{d_c} = \frac{1}{2c_f} \left(1 - \sqrt{1 - \frac{14c_f f_y}{c_1 \beta^3} \left(\frac{\kappa \ell}{d_c} \right)^2} \right) \quad (\text{A.3.16})$$

For small axial restraint ($c < 0.05$) the critical deformation may be taken as

$$\frac{w}{d_c} = \frac{3.5f_y}{c_1\beta^3} \left(\frac{\kappa\ell}{d_c} \right)^2 \quad (\text{A.3.17})$$

Stiffened plates/ I/H-profiles:

In lieu of more accurate calculations the expressions given for circular profiles in Eq. (A.3.16) and (A.3.17) may be used with

$$\begin{aligned} d_c &= \text{characteristic dimension for local buckling, equal to twice the distance from the} \\ &= h \quad \text{plastic neutral axis in bending to the extreme fibre of the cross-section} \\ &= 2h_w \quad \text{height of cross-section for symmetric I –profiles} \\ &\quad \text{for stiffened plating (for simplicity)} \end{aligned}$$

For flanges subjected to compression;

$$\beta = 2.5 \frac{b_f/t_f}{\sqrt{235/f_y}} \quad \text{class 1 cross-sections} \quad (\text{A.3.18})$$

$$\beta = 3 \frac{b_f/t_f}{\sqrt{235/f_y}} \quad \text{class 2 and class 3 cross-sections} \quad (\text{A.3.19})$$

For webs subjected to bending

$$\beta = 0.7 \frac{h_w/t_w}{\sqrt{235/f_y}} \quad \text{class 1 cross-sections} \quad (\text{A.3.20})$$

$$\beta = 0.8 \frac{h_w/t_w}{\sqrt{235/f_y}} \quad \text{class 2 and class 3 cross-sections} \quad (\text{A.3.21})$$

$$\begin{aligned} b_f &= \text{flange width} \\ t_f &= \text{flange thickness} \\ h_w &= \text{web height} \\ t_w &= \text{web thickness} \end{aligned}$$

A.3.10.3 Lateral stability at yield hinges

The compressed part of I or H beams needs to be laterally supported to avoid instability. Unless more accurate investigations are undertaken, H and I beams may be considered to be stable when a yield hinge is formed if the length between lateral supports are less than:

$$L \leq 0,2 * b_f \sqrt{\frac{EA_f}{f_y(A_f + 0.33A_w)}} \quad (\text{A.3.22})$$

$$\begin{aligned} b_f &= \text{flange width} \\ A_f &= \text{flange area} \\ A_w &= \text{web area} \end{aligned}$$

A.3.10.4 Tensile Fracture

The degree of plastic deformation or critical strain at fracture will show a significant scatter and depends upon the following factors:

- material toughness
- presence of defects
- strain rate
- presence of strain concentrations

The critical strain for plastic deformations of sections containing defects need to be determined based on fracture mechanics methods. (See chapter 6.5.) Welds normally contain defects and welded joints are likely to achieve lower toughness than the parent material. For these reasons structures that need to undergo large plastic deformations should be designed in such a way that the plastic straining takes place away outside the weld. In ordinary full penetration welds, the overmatching weld material will ensure that minimal plastic straining occurs in the welded joints even in cases with yielding of the gross cross section of the member. In such situations, the critical strain will be in the parent material and will be dependent upon the following parameters:

- stress gradients
- dimensions of the cross section
- presence of strain concentrations
- material yield to tensile strength ratio
- material ductility

Simple plastic theory does not provide information on strains as such. Therefore, strain levels should be assessed by means of adequate analytic models of the strain distributions in the plastic zones or by non-linear finite element analysis with a sufficiently detailed mesh in the plastic zones.

When structures are designed so that yielding take place in the parent material, the following value for the critical average strain in axially loaded plate material may be used in conjunction with nonlinear finite element analysis or simple plastic analysis

$$\varepsilon_{cr} = \left(0.02 + 0.65 \frac{t}{\ell} \right) \frac{355}{f_y} \quad (\text{A.3.23})$$

where:

- t = plate thickness
- ℓ = length of plastic zone. Minimum 5t
- f_y = yield stress in MPa

A.3.10.5 Tensile fracture in yield hinges

When the force deformation relationships for beams given in Section A.3.7.2 are used rupture may be assumed to occur when the deformation exceeds a value given by

$$\frac{w}{d_c} = \frac{c_1}{2c_f} \left(\sqrt{1 + \frac{4c_w c_f \varepsilon_{cr}}{c_1}} - 1 \right) \quad (\text{A.3.24})$$

where the following factors are defined;

Displacement factor

$$c_w = \frac{1}{c_1} \left(c_{lp} \left(1 - \frac{1}{3} c_{lp} \right) + 4 \left(1 - \frac{W}{W_p} \right) \frac{\varepsilon_y}{\varepsilon_{cr}} \right) \left(\frac{\kappa \ell}{d_c} \right)^2 \quad (\text{A.3.25})$$

plastic zone length factor

$$c_{lp} = \frac{\left(\frac{\varepsilon_{cr}}{\varepsilon_y} - 1 \right) \frac{W}{W_p} H}{\left(\frac{\varepsilon_{cr}}{\varepsilon_y} - 1 \right) \frac{W}{W_p} H + 1} \quad (\text{A.3.26})$$

axial flexibility factor

$$c_f = \left(\frac{\sqrt{c}}{1 + \sqrt{c}} \right)^2 \quad (\text{A.3.27})$$

non-dimensional plastic stiffness

$$H = \frac{E_p}{E} = \frac{1}{E} \left(\frac{f_{cr} - f_y}{\varepsilon_{cr} - \varepsilon_y} \right) \quad (\text{A.3.28})$$

- $c_1 = 2$ for clamped ends
- $c_1 = 1$ for pinned ends
- $c =$ non-dimensional spring stiffness, refer Section A.3.7.2
- $\kappa/l \leq 0.5/$ the smaller distance from location of collision load to adjacent joint
- $W =$ elastic section modulus
- $W_p =$ plastic section modulus
- $\varepsilon_{cr} =$ critical strain for rupture
- $\varepsilon_y = \frac{f_y}{E}$ yield strain
- $f_y =$ yield strength
- $f_{cr} =$ strength corresponding to ε_{cr}

The characteristic dimension shall be taken as:

- $d_c = D$ diameter of tubular beams
- $d_c = 2h_w$ twice the web height for stiffened plates
- $d_c = h$ height of cross-section for symmetric I-profiles

For small axial restraint ($c < 0.05$) the critical deformation may be taken as

$$\frac{w}{d_c} = c_w \varepsilon_{cr} \quad (\text{A.3.29})$$

The critical strain ε_{cr} and corresponding strength f_{cr} should be selected so that idealised bi-linear stress-strain relation gives reasonable results. See Commentary. For typical steel material grades the following values are proposed:

Table A.3-4 Proposed values for ε_{cr} and H for different steel grades

Steel grade	ε_{cr}	H
S 235	20 %	0.0022
S 355	15 %	0.0034
S 460	10 %	0.0034

A.3.11 Resistance of large diameter, stiffened columns

A.3.11.1 General

Impact on a ring stiffener as well as midway between ring stiffeners shall be considered.
Plastic methods of analysis are generally applicable.

A.3.11.2 Longitudinal stiffeners

For ductile design the resistance of longitudinal stiffeners in the beam mode of deformation can be calculated using the procedure described for stiffened plating, section A.3.7.

For strength design against stern corner impact, the plastic bending moment capacity of the longitudinal stiffeners has to comply with the requirement given in Figure A.3-11, on the assumption that the entire load given in Table A.3-1 is taken by one stiffener.

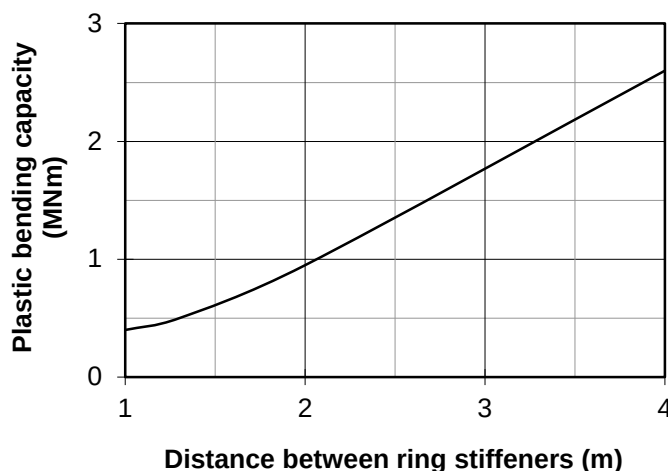


Figure A.3-11 Required bending capacity of longitudinal stiffeners

A.3.11.3 Ring stiffeners

In lieu of more accurate analysis the plastic collapse load of a ring-stiffener can be estimated from:

$$F_0 = \frac{4\sqrt{2}M_p}{\sqrt{w_c}D} \quad (\text{A.3.30})$$

where

$$w_c = \frac{W_p}{A_e} = \text{characteristic deformation of ring stiffener}$$

D = column radius
 M_p = plastic bending resistance of ring-stiffener including effective shell flange
 W_p = plastic section modulus of ring stiffener including effective shell flange
 A_e = area of ring stiffener including effective shell flange

Effective flange of shell plating: Use effective flange of stiffened plates, see Chapter 6.

For ductile design it can be assumed that the resistance of the ring stiffener is constant and equal to the plastic collapse load, provided that requirements for stability of cross-sections are complied with, refer Section A.3.10.2.

A.3.11.4 Decks and bulkheads

Calculation of energy dissipation in decks and bulkheads has to be based upon recognised methods for plastic analysis of deep, axial crushing. It shall be documented that the collapse mechanisms assumed yield a realistic representation of the true deformation field.

A.3.12 Energy dissipation in floating production vessels

For strength design the side or stern shall resist crushing force of the bow of the off-take tanker. In lieu of more accurate calculations the force-deformation curve given in Section A.3.5.2 may be applied.

For ductile design the resistance of stiffened plating in the beam mode of deformation can be calculated using the procedure described in section A.3.7.2.

Calculation of energy dissipation in stringers, decks and bulkheads subjected to gross, axial crushing shall be based upon recognised methods for plastic analysis, eg. /3/ and /5/. It shall be documented that the folding mechanisms assumed yield a realistic representation of the true deformation field.

A.3.13 Global integrity during impact

Normally, it is unlikely that the installation will turn into a global collapse mechanism under direct collision load, because the collision load is typically an order of magnitude smaller than the resultant design wave force.

Linear analysis often suffices to check that global integrity is maintained.

The installation should be checked for the maximum collision force.

For installations responding predominantly statically the maximum collision force occurs at maximum deformation.

For structures responding predominantly impulsively the maximum collision force occurs at small global deformation of the platform. An upper bound to the collision force is to assume that the installation is fixed with respect to global displacement. (e.g. jack-up fixed with respect to deck displacement)

A.4 DROPPED OBJECTS

A.4.1 General

The dropped object action is characterised by a kinetic energy, governed by the mass of the object, including any hydrodynamic added mass, and the velocity of the object at the instant of impact. In most cases the major part of the kinetic energy has to be dissipated as strain energy in the impacted component and, possibly, in the dropped object. Generally, this involves large plastic strains and significant structural damage to the impacted component. The strain energy dissipation is estimated from force-deformation relationships for the component and the object, where the deformations in the component shall comply with ductility and stability requirements.

The load bearing function of the installation shall remain intact with the damages imposed by the dropped object action. In addition, the residual strength requirements given in Section A.7 shall be complied with.

Dropped objects are rarely critical to the global integrity of the installation and will mostly cause local damages. The major threat to global integrity is probably puncturing of buoyancy tanks, which could impair the hydrostatic stability of floating installations. Puncturing of a single tank is normally covered by the general requirements to compartmentation and watertight integrity given in ISO 19900 and NORSOK N-001.

The structural effects from dropped objects may either be determined by non-linear dynamic finite element analyses or by energy considerations combined with simple elastic-plastic methods as given in Sections A.4.2 - A.4.5.

If non-linear dynamic finite element analysis is applied all effects described in the following paragraphs shall either be implicitly covered by the modelling adopted or subjected to special considerations, whenever relevant.

A.4.2 Impact velocity

The kinetic energy of a falling object is given by

$$E_{\text{kin}} = \frac{1}{2}mv^2 \quad (\text{A.4.1})$$

for objects falling in air and,

$$E_{\text{kin}} = \frac{1}{2}(m + a)v^2 \quad (\text{A.4.2})$$

for objects falling in water.

a = hydrodynamic added mass for considered motion

For impacts in air the velocity is given by

$$v = \sqrt{2gs} \quad (\text{A.4.3})$$

s = travelled distance from drop point
 v = v_0 at sea surface

For objects falling rectilinearly in water the velocity depends upon the reduction of speed during impact with water and the falling distance relative to the characteristic distance for the object.

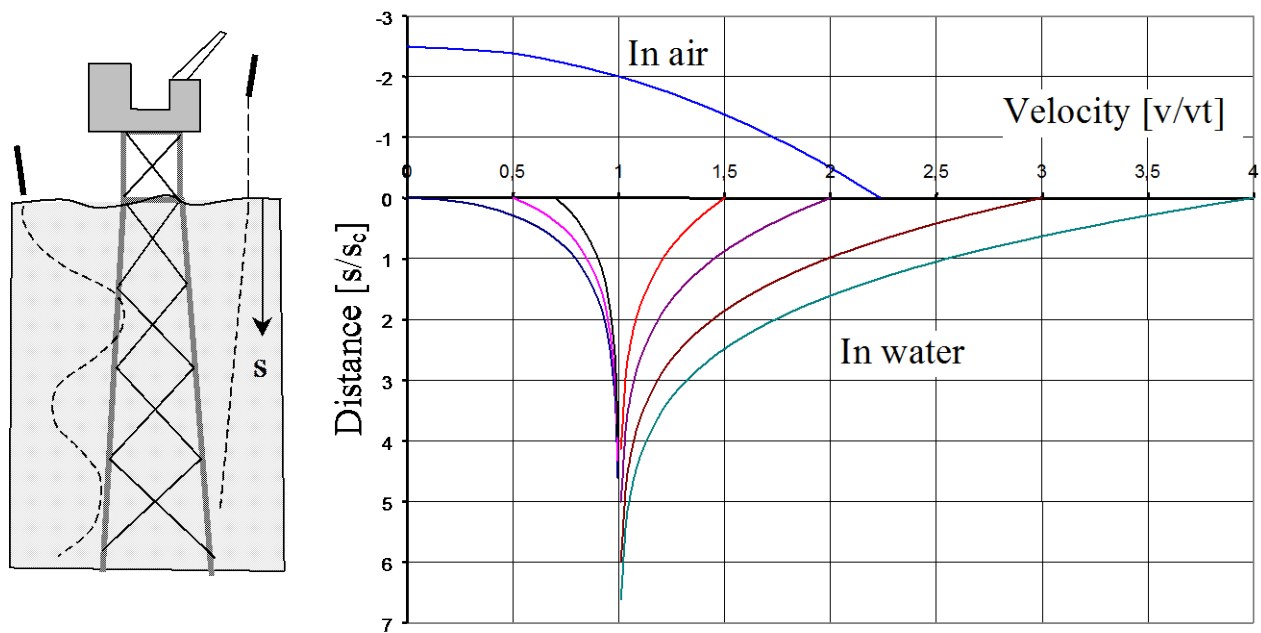


Figure A.4-1 Velocity profile for objects falling in water

The loss of momentum during impact with water is given by

$$m\Delta v = \int_0^{t_d} F(t)dt \quad (\text{A.4.4})$$

$F(t)$ = force during impact with sea surface

After the impact with water the object proceeds with the speed

$$v = v_0 - \Delta v$$

Assuming that the hydrodynamic resistance during fall in water is of drag type the velocity in water can be taken from Figure A.4-1 where

$$v_t = \sqrt{\frac{2g(m - \rho_w V)}{\rho_w C_d A_p}} \quad = \text{terminal velocity for the object}$$

$$s_c = \frac{m + a}{\rho_w C_d A_p} = \frac{v_t^2 \left(1 + \frac{a}{m}\right)}{2g \left(1 - \frac{\rho_w V}{m}\right)} \quad = \text{characteristic distance}$$

ρ_w = density of sea water
 C_d = hydrodynamic drag coefficient for the object in the considered motion
 m = mass of object
 A_p = projected cross-sectional area of the object
 V = object displacement

The major uncertainty is associated with calculating the loss of momentum during impact with sea surface, given by Equation (A.4.4). However, if the travelled distance is such that the velocity is close to the terminal velocity, the impact with sea surface is of little significance.

Typical terminal velocities for some typical objects are given in Table A.4-1

Table A.4-1 Terminal velocities for objects falling in water.

Item	Mass [kN]	Terminal velocity [m/s]
Drill collar	28	23-24
Winch,	250	
Riser pump	100	
BOP annular preventer	50	16
Mud pump	330	7

Rectilinear motion is likely for blunt objects and objects which do not rotate about their longitudinal axis. Bar-like objects (e.g. pipes) which do not rotate about their longitudinal axis may execute lateral, damped oscillatory motions as illustrated in Figure A.4-1.

A.4.3 Dissipation of strain energy

The structural response of the dropped object and the impacted component can formally be represented as load-deformation relationships as illustrated in Figure A.4-2. The part of the impact energy dissipated as strain energy equals the total area under the load-deformation curves.

$$E_s = E_{s,o} + E_{s,i} = \int_0^{w_{o,max}} R_o dw_o + \int_0^{w_{i,max}} R_i dw_i \quad (\text{A.4.5})$$

As the load level is not known a priori an incremental approach is generally required.

Often the object can be assumed to be infinitely rigid (e.g axial impact from pipes and massive objects) so that all energy is to be dissipated by the impacted component..

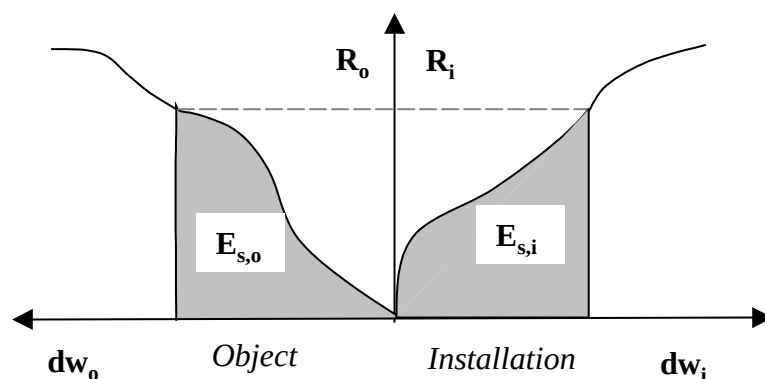


Figure A.4-2 Dissipation of strain energy in dropped object and installation

If the object is assumed to be deformable, the interactive nature of the deformation of the two structures should be recognised.

A.4.4 Resistance/energy dissipation

A.4.4.1 Stiffened plates subjected to drill collar impact

The energy dissipated in the plating subjected to drill collar impact is given by

$$E_{sp} = \frac{R^2}{2K} \left(1 + 0.48 \frac{m_i}{m} \right)^2 \quad (\text{A.4.6})$$

$$K = \frac{1}{2} \pi f_y t \left(\frac{1 + 5 \frac{d}{r} - 6c^2 + 6.25 \left(\frac{d}{2r} \right)^2}{(1+c)^2} \right) \quad : \text{stiffness of plate enclosed by hinge circle}$$

f_y = characteristic yield strength

$$c = -e^{-2.5 \left(1 - \frac{d}{2r} \right)}$$

$R = \pi d t \tau$ = contact force for $\tau \leq \tau_{cr}$ refer Section A.4.5.1 for τ_{cr}

$m_i = \rho_p \pi r^2 t$ = mass of plate enclosed by hinge circle

m = mass of dropped object

ρ_p = mass density of steel plate

d = smaller diameter at threaded end of drill collar

r = smaller distance from the point of impact to the plate boundary defined by adjacent stiffeners/girders, refer Figure A.4-3.

For validity range of design formula see Commentary.

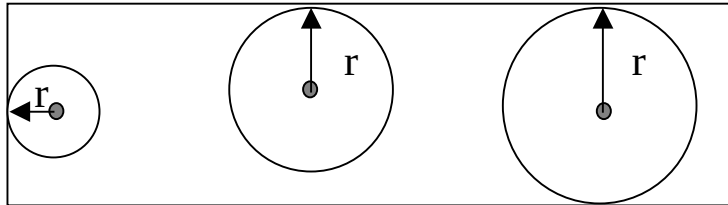


Figure A.4-3 Definition of distance to plate boundary

A.4.4.2 Stiffeners/girders

In lieu of more accurate calculations stiffeners and girders subjected to impact with blunt objects may be analysed with resistance models given in Section A.6.9.

A.4.4.3 Dropped object

Calculation of energy dissipation in deformable dropped objects shall be based upon recognised methods for plastic analysis. It shall be documented that the collapse mechanisms assumed yield a realistic representation of the true deformation field.

A.4.5 Limits for energy dissipation

A.4.5.1 Pipes on plated structures

The maximum shear stress for plugging of plates due to drill collar impacts may be taken as

$$\tau_{cr} = f_u \left(0.42 + 0.41 \frac{t}{d} \right) \quad (\text{A.4.7})$$

f_u = ultimate material tensile strength

A.4.5.2 Blunt objects

For stability of cross-sections and tensile fracture, refer Section A.3.10.

A.5 FIRE

A.5.1 General

The characteristic fire structural action is temperature rise in exposed members. The temporal and spatial variation of temperature depends on the fire intensity, whether or not the structural members are fully or partly engulfed by the flame and to what extent the members are insulated.

Structural steel expands at elevated temperatures and internal stresses are developed in redundant structures. These stresses are most often of moderate significance with respect to global integrity. The heating causes also progressive loss of strength and stiffness and is, in redundant structures, accompanied by redistribution of forces from members with low strength to members that retain their load bearing capacity. A substantial loss of load-bearing capacity of individual members and subassemblies may take place, but the load bearing function of the installation shall remain intact during exposure to the fire action.

In addition, the residual strength requirements given in Section A.7 shall be complied with.

Structural analysis may be performed on either

- individual members
- subassemblies
- entire system

The assessment of fire load effect and mechanical response shall be based on either

- simple calculation methods applied to individual members,
- general calculation methods,

or a combination.

Simple calculation methods may give overly conservative results. General calculation methods are methods in which engineering principles are applied in a realistic manner to specific applications.

Assessment of individual members by means of simple calculation methods should be based upon the provisions given in Eurocode 3 Part 1.2. /2/.

Assessment by means of general calculation methods shall satisfy the provisions given in Eurocode 3 Part 1.2 , Section 4.3.

In addition, the requirements given in this section for mechanical response analysis with nonlinear finite element methods shall be complied with.

Assessment of ultimate strength is not needed if the maximum steel temperature is below 400 °C, but deformation criteria may have to be checked for impairment of main safety functions.

A.5.2 General calculation methods

Structural analysis methods for non-linear, ultimate strength assessment may be classified as

- stress-strain based methods
- stress-resultants based (yield/plastic hinge) methods

Stress-strain based methods are methods where non-linear material behaviour is accounted for on fibre level.

Stress-resultants based methods are methods where non-linear material behaviour is accounted for on stress-resultants level based upon closed form solutions/interaction equation for cross-sectional forces and moments.

A.5.3 Material modelling

In stress-strain based analysis temperature dependent stress-strain relationships given in Eurocode 3, Part 1.2 , Section 3.2 may be used.

For stress resultants based design the temperature reduction of the elastic modulus may be taken as $k_{E,\theta}$ according to Eurocode 3 . The yield stress may be taken equal to the effective yield stress, $f_{y,\theta}$. The temperature reduction of the effective yield stress may be taken as $k_{y,\theta}$.

Provided that above the above requirements are complied with creep does need explicit consideration.

A.5.4 Equivalent imperfections

To account for the effect of residual stresses and lateral distortions compressive members shall be modelled with an initial, sinusoidal imperfection with amplitude given by

Elasto-plastic material models

$$\frac{e^*}{\ell} = \frac{\alpha}{\pi} \sqrt{\frac{\sigma_Y}{E}} \frac{i}{z_0}$$

Elastic-plastic material models:

$$\frac{e^*}{\ell} = \frac{W_p}{W} \frac{\alpha}{\pi} \sqrt{\frac{\sigma_Y}{E}} \frac{i}{z_0} = \frac{\alpha}{\pi} \sqrt{\frac{\sigma_Y}{E}} \frac{W_p}{\sqrt{AI}}$$

α	=	0.5 for fire exposed members according to column curve c, Eurocode 3
i	=	radius of gyration
z_0	=	distance from neutral axis to extreme fibre of cross-section
W_p	=	plastic section modulus
W	=	elastic section modulus
A	=	cross-sectional area
I	=	moment of inertia
e^*	=	amplitude of initial distortion
ℓ	=	member length

The initial out-of-straightness should be applied on each physical member. If the member is modelled by several finite elements the initial out-of-straightness should be applied as displaced nodes.

The initial out-of-straightness shall be applied in the same direction as the deformations caused by the temperature gradients.

A.5.5 Empirical correction factor

The empirical correction factor of 1.2 has to be accounted for in calculating the critical strength in compression and bending for design according to Eurocode 3, refer Comm. A.5.5.

A.5.6 Local cross sectional buckling

If shell modelling is used, it shall be verified that the software and the modelling is capable of predicting local buckling with sufficient accuracy. If necessary, local shell imperfections have to be introduced in a similar manner to the approach adopted for lateral distortion of beams

If beam modelling is used local cross-sectional buckling shall be given explicit consideration.

In lieu of more accurate analysis cross-sections subjected to plastic deformations shall satisfy compactness requirements given in Eurocode 3 :

- class 1 : Locations with plastic hinges (approximately full plastic utilization)
- class 2 : Locations with yield hinges (partial plastification)

If this criterion is not complied with explicit considerations shall be performed. The load-bearing capacity will be reduced significantly after the onset of buckling, but may still be significant. A conservative approach is to remove the member from further analysis.

Compactness requirements for class 1 and class 2 cross-sections may be disregarded provided that the member is capable of developing significant membrane forces.

A.5.7 Ductility limits

A.5.7.1 General

The ductility of beams and connections increase at elevated temperatures compared to normal conditions. Little information exists.

A.5.7.2 Beams in bending

In lieu of more accurate analysis requirements given in Section A.3.10 are to be complied with.

A.5.7.3 Beams in tension

In lieu of more accurate analysis an average elongation of 3% of the member length with a reasonably uniform temperature can be assumed.

Local temperature peaks may localise plastic strains. It is considered to be to the conservative side to use critical strains for steel under normal temperatures. See A.3.10.4 and A.3.10.5.

A.5.8 Capacity of connections

In lieu of more accurate calculations the capacity of the connection can be taken as:

$$R_{\theta} = k_{y,\theta} R_0$$

where

R_0	=	capacity of connection at normal temperature
$k_{y,\theta}$	=	temperature reduction of effective yield stress for maximum temperature in connection

A.6 EXPLOSIONS

A.6.1 General

Explosion loads are characterised by temporal and spatial pressure distribution. The most important temporal parameters are rise time, maximum pressure and pulse duration.

For components and sub-structures the explosion pressure shall normally be considered uniformly distributed. On global level the spatial distribution is normally nonuniform both with respect to pressure and duration.

The response to explosion loads may either be determined by non-linear dynamic finite element analysis or by simple calculation models based on Single Degree Of Freedom (SDOF) analogies and elastic-plastic methods of analysis.

If non-linear dynamic finite element analysis is applied all effects described in the following paragraphs shall either be implicitly covered by the modelling adopted or subjected to special considerations, whenever relevant

In the simple calculation models the component is transformed to a single spring-mass system exposed to an equivalent load pulse by means of suitable shape functions for the displacements in the elastic and elastic-plastic range. The shape functions allow calculation of the characteristic resistance curve and equivalent mass in the elastic and elastic-plastic range as well as the fundamental period of vibration for the SDOF system in the elastic range.

Provided that the temporal variation of the pressure can be assumed to be triangular, the maximum displacement of the component can be calculated from design charts for the SDOF system as a function of pressure duration versus fundamental period of vibration and equivalent load amplitude versus maximum resistance in the elastic range. The maximum displacement must comply with ductility and stability requirements for the component.

The load bearing function of the installation shall remain intact with the damages imposed by the explosion loads. In addition, the residual strength requirements given in Section A.7 shall be complied with.

A.6.2 Classification of response

The response of structural components can conveniently be classified into three categories according to the duration of the explosion pressure pulse, t_d , relative to the fundamental period of vibration of the component, T :

- Impulsive domain - $t_d/T < 0.3$
- Dynamic domain - $0.3 < t_d/T < 3$
- Quasi-static domain - $3 < t_d/T$

Impulsive domain:

The response is governed by the impulse defined by

$$I = \int_0^{t_d} F(t) dt \quad (\text{A.6.1})$$

Hence, the structure may resist a very high peak pressure provided that the duration is sufficiently small. The maximum deformation, $w_{\max}$, of the component can be calculated iteratively from the equation

$$I = \sqrt{2m_{\text{eq}} \int_0^{w_{\max}} R(w) dw} \quad (\text{A.6.2})$$

where

$R(w)$ = force-deformation relationship for the component
 m_{eq} = equivalent mass for the component

Quasi-static-domain:

The response is governed by the peak pressure and the rise time of the pressure relative to the fundamental period of vibration. If the rise time is small the maximum deformation of the component can be solved iteratively from the equation:

$$w_{\max} = \frac{1}{F_{\max}} \int_0^{w_{\max}} R(w) dw \quad (\text{A.6.3})$$

If the rise time is large the maximum deformation can be solved from the static condition

$$F_{\max} = R(w_{\max}) \quad (\text{A.6.4})$$

Dynamic domain:

The response has to be solved from numerical integration of the dynamic equations of equilibrium.

A.6.3 Failure modes for stiffened panels

Various failure modes for a stiffened panel are illustrated in Figure A.6-1. Suggested analysis model and reference to applicable resistance functions are listed in Table A.6-1. Application of a Single Degree of Freedom (SDOF) model in the analysis of stiffeners/girders with effective flange is implicitly based on the assumption that dynamic interaction between the plate flange and the profile can be neglected.

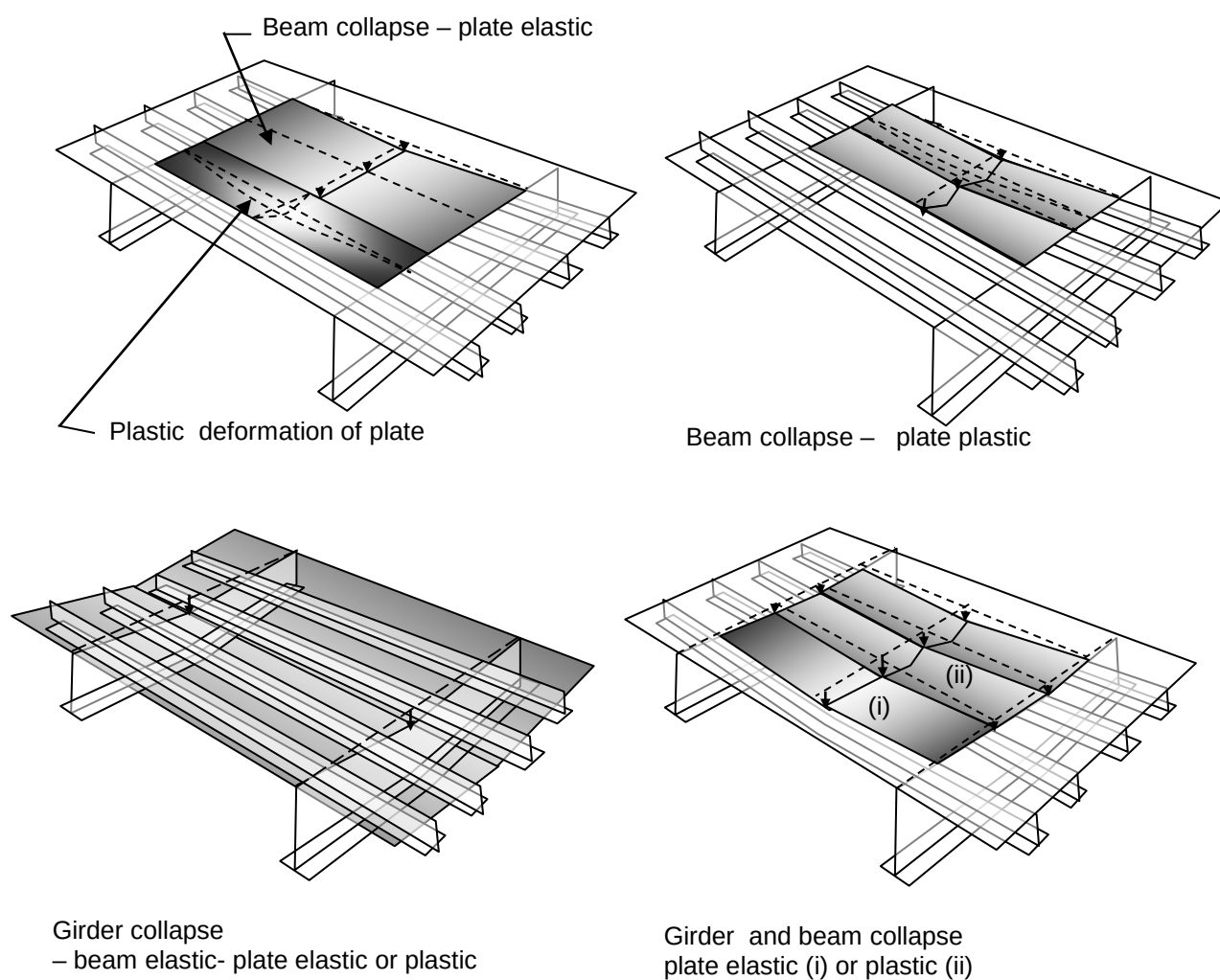


Figure A.6-1 Failure modes for two-way stiffened panel

Table A.6-1 Analysis models

Failure mode	Simplified analysis model	Resistance models	Comment
Elastic-plastic deformation of plate	SDOF	Section A.6.8	
Stiffener plastic – plate elastic	SDOF	Stiffener: Section A.6.9.1-2. Plate: Section A.6.8.1	Elastic, effective flange of plate
Stiffener plastic – plate plastic	SDOF	Stiffener: Section A.6.9.1-2. Plate: Section A.6.8	Effective width of plate at mid span. Elastic, effective flange of plate at ends.
Girder plastic – stiffener and plating elastic	SDOF	Girder: Section A.6.9.1-2 Plate: Section A.6.8	Elastic, effective flange of plate with concentrated loads (stiffener reactions). Stiffener mass included.
Girder plastic – stiffener elastic – plate plastic	SDOF	Girder: Section A.6.9.1-2 Plate: Section A.6.8	Effective width of plate at girder mid span and ends. Stiffener mass included
Girder and stiffener plastic – plate elastic	MDOF	Girder and stiffener: Section A.6.9.1-2 Plate: Section A.6.8	Dynamic reactions of stiffeners → loading for girders
Girder and stiffener plastic – plate plastic	MDOF	Girder and stiffener: Section A.6.9.1-2 Plate: Section A.6.8	Dynamic reactions of stiffeners → loading for girders

By girder/stiffener **plastic** is understood that the maximum displacement $w_{\max}$ exceeds the elastic limit w_{el}

A.6.4 SDOF system analogy

A.6.4.1 General

Biggs method:

For many practical design problems it is convenient to assume that the structure - exposed to the dynamic pressure pulse - ultimately attains a deformed configuration comparable to the static deformation pattern. Using the static deformation pattern as displacement shape function, i.e.

$$w(x, t) = \phi(x)y(t)$$

the dynamic equations of equilibrium can be transformed to an equivalent single degree of freedom system:

$$\bar{m}\ddot{y} + \bar{k}y = \bar{f}(t)$$

$$\phi(x) = \text{displacement shape function}$$

$$y(t) = \text{displacement amplitude}$$

$$\bar{m} = \int_{\ell} m\phi(x)^2 dx + \sum_i M_i \phi_i^2 = \text{generalized mass}$$

$$\bar{f}(t) = \int_{\ell} q(t)\phi(x)dx + \sum_i F_i \phi_i = \text{generalized load}$$

$$\bar{k} = \int_{\ell} EI\phi_{,xx}(x)^2 dx = \text{generalized elastic bending stiffness}$$

$$\bar{k} = 0 = \text{generalized plastic bending stiffness (fully developed mechanism)}$$

$$\bar{k} = \int_{\ell} N\phi_{,x}(x)^2 dx = \text{generalized membrane stiffness (fully plastic: } N = N_p)$$

$$m = \text{distributed mass}$$

$$M_i = \text{concentrated mass}$$

$$q = \text{explosion load}$$

$$F_i = \text{concentrated load (e.g. support reactions)}$$

$$x_i = \text{position of concentrated mass/load}$$

$$\varphi_i = \varphi(x = x_i)$$

The equilibrium equation can alternatively be expressed as:

$$(K_{lm,u}M_u + K_{lm,c}M_c)\ddot{y} + k(y)y = F(t) \quad (\text{A.6.5})$$

where

$$K_{lm,u} = \frac{K_{m,u}}{K_l} = \text{load-mass transformation factor for uniform mass}$$

$$K_{lm,c} = \frac{K_{m,c}}{K_l} = \text{load-mass transformation factor for concentrated mass}$$

$$K_{m,u} = \frac{\int m\varphi(x)^2 dx}{M_u} = \text{mass transformation factor for uniform mass}$$

$$K_{m,c} = \frac{\sum M_i \varphi_i^2}{M_c} = \text{mass transformation factor for concentrated mass}$$

$$K_l = \frac{\int q\varphi(x)dx}{F} = \text{load transformation factor for uniformly distributed load}$$

$$K_l = \frac{\sum F_i \varphi_i}{F} = \text{load transformation factor for concentrated load}$$

$$M_u = \int_{\ell} m dx = \text{total uniformly, distributed mass}$$

$$M_c = \sum_i M_i = \text{total concentrated mass}$$

$$F = \int_{\ell} q dx = \text{total load in case of uniformly distributed load}$$

$$F = \sum_i F_i = \text{total load in case of concentrated load}$$

$$k_e = \frac{\bar{k}}{k_l} = \text{equivalent stiffness}$$

The natural period of vibration for the equivalent system in the linear resistance domain is given by

$$T = 2\pi \sqrt{\frac{\bar{m}}{\bar{k}}} = 2\pi \sqrt{\frac{K_{lm,u}M_u + K_{lm,c}M_c}{k_e}} \quad (\text{A.6.6})$$

The response , $y(t)$, is - in addition to the load history - entirely governed by the total mass, load-mass factor and the equivalent stiffness.

For a *linear system*, the load mass factor and the equivalent stiffness are constant $k_1 = k_e$. The response is then alternatively governed by the eigenperiod and the equivalent stiffness.

For a *non-linear system*, the load-mass factor and the equivalent stiffness depend on the response (deformations). Non-linear systems may often conveniently be approximated by equivalent bi-linear or tri-linear systems, see Section A.6.7. In such cases the response can be expressed in terms of (see Figure A.6-10 for definitions):

$$k_1 = \text{equivalent stiffness in the initial, linear resistance domain}$$

$$y_{el} = \text{displacement at the end of the initial, linear resistance domain}$$

- T = eigenperiod in the initial, linear resistance domain
and, if relevant,
 k_3 = normalised equivalent resistance in the third linear resistance domain.

Equivalent stiffnesses are given explicitly or can be derived from load-deformation relationships given in Section A.6.9. If the response is governed by different mechanical behaviour relevant equivalent stiffnesses must be calculated.

For a given explosion load history the maximum displacement, $y_{\max}$, is found by analytical or numerical integration of equation (A.6.6).

For standard load histories and standard resistance curves maximum displacements can be presented in design charts. Figure A.6-2 shows the normalised maximum displacement of a SDOF system with a bi- ($k_3=0$) or trilinear ($k_3 > 0$) resistance function, exposed to a triangular pressure pulse with zero rise time. When the duration of the pressure pulse relative to the eigenperiod in the initial, linear resistance range is known, the maximum displacement can be determined directly from the diagram as illustrated in Figure A.6-2.

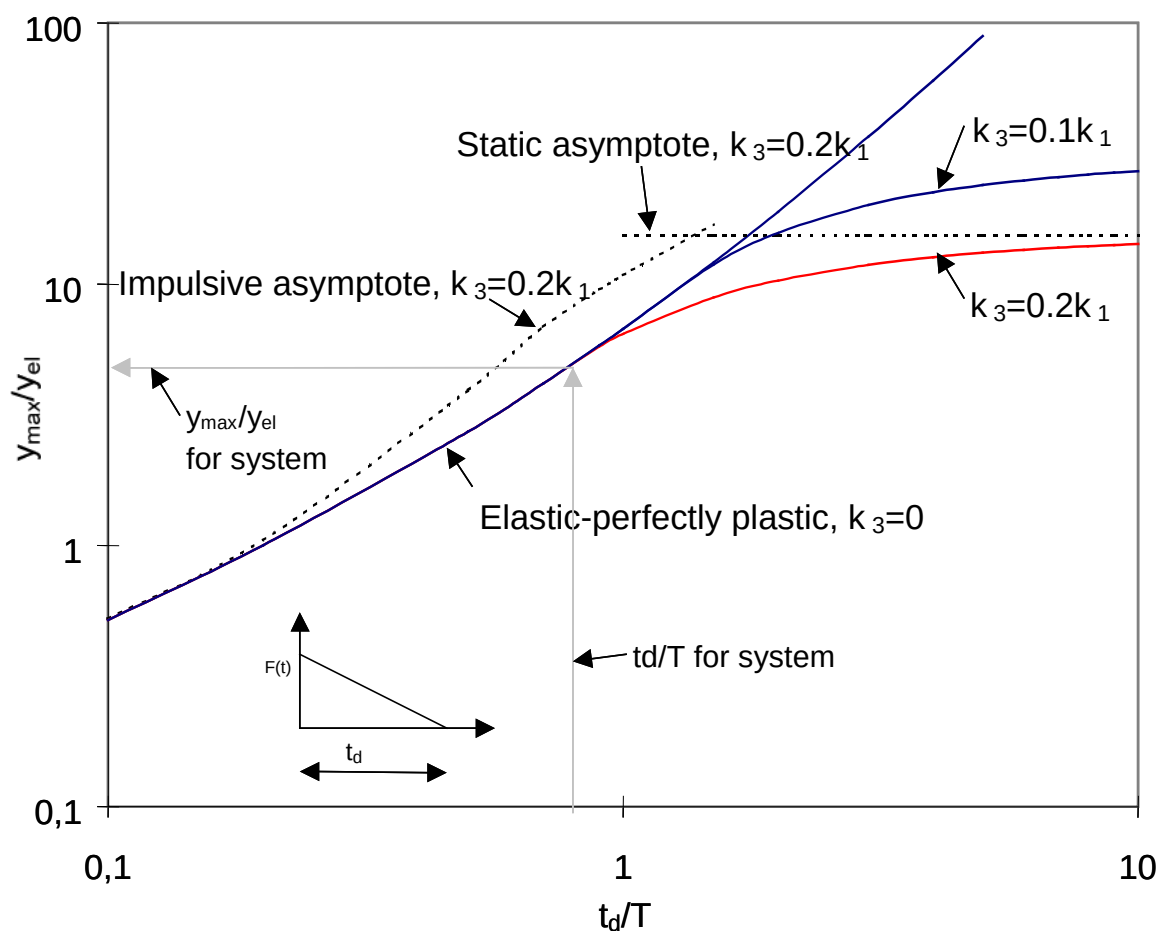


Figure A.6-2 Maximum response a SDOF system to a triangular pressure pulse with zero rise time.
 $F_{\max}/R_{el} = 2$

Design charts for systems with bi- or tri-linear resistance curves subjected to a triangular pressure pulse with different rise time are given in Figures A.6-4 - A6-7.

Baker's method

The governing equations (A.6.1) and (A.6.2) for the maximum response in the impulsive domain and the quasi-static domain may also be used along with response charts for maximum displacement for different $F_{\max}/R_{el}$ ratios to produce pressure-impulse ($F_{\max}, I$) diagrams - iso-damage curves - provided that the

maximum pressure is known. Figure A.6-3 shows such a relationship obtained for an elastic-perfectly plastic system when the maximum dynamic response is $y_{\max}/y_{el}=10$.

Pressure-impulse combinations to the left and below of the iso-damage curve represent admissible events, to the right and above inadmissible events.

The advantage of using iso-damage diagrams is that "back-ward" calculations are possible: The diagram is established on the basis of the resistance curve. The information may be used to screen explosion pressure histories and eliminate those that obviously lie in the admissible domain. This will reduce the need for large complex simulation of explosion scenarios.

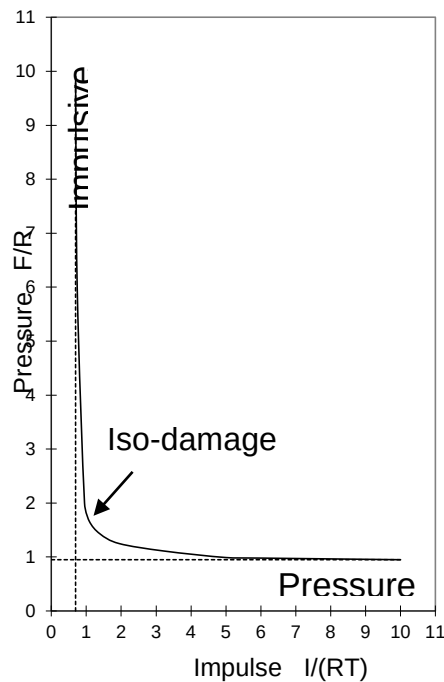


Figure A.6-3 Iso-damage curve for $y_{\max}/y_{el} = 10$. Triangular pressure pulse.

A.6.4.2 Dynamic response charts for SDOF system

Transformation factors for elastic-plastic-membrane deformation of beams and one-way slabs with different boundary conditions are given in Table A.6-2

Maximum displacement for a SDOF system exposed to different pressure histories are displayed in Figures A.6-4 - A.6-7.

The response of the system is based upon the resistance in the linear range, $k_e=k_1$, where the equivalent stiffness is determined from the elastic solution to the actual system.

If the displacement shape function changes as a non-linear structure undergoes deformation the transformation factors change. In lieu of accurate analysis an average value of the combined load-mass transformation factor can be used:.

$$K_{lm}^{average} = \frac{K_{lm}^{elastic} + (\mu - 1)K_{lm}^{plastic}}{\mu} \quad (A.6.7)$$

$$\mu = y_{\max}/y_{el} \quad \text{ductility ratio}$$

Since μ is not known a priori iterative calculations may be necessary.

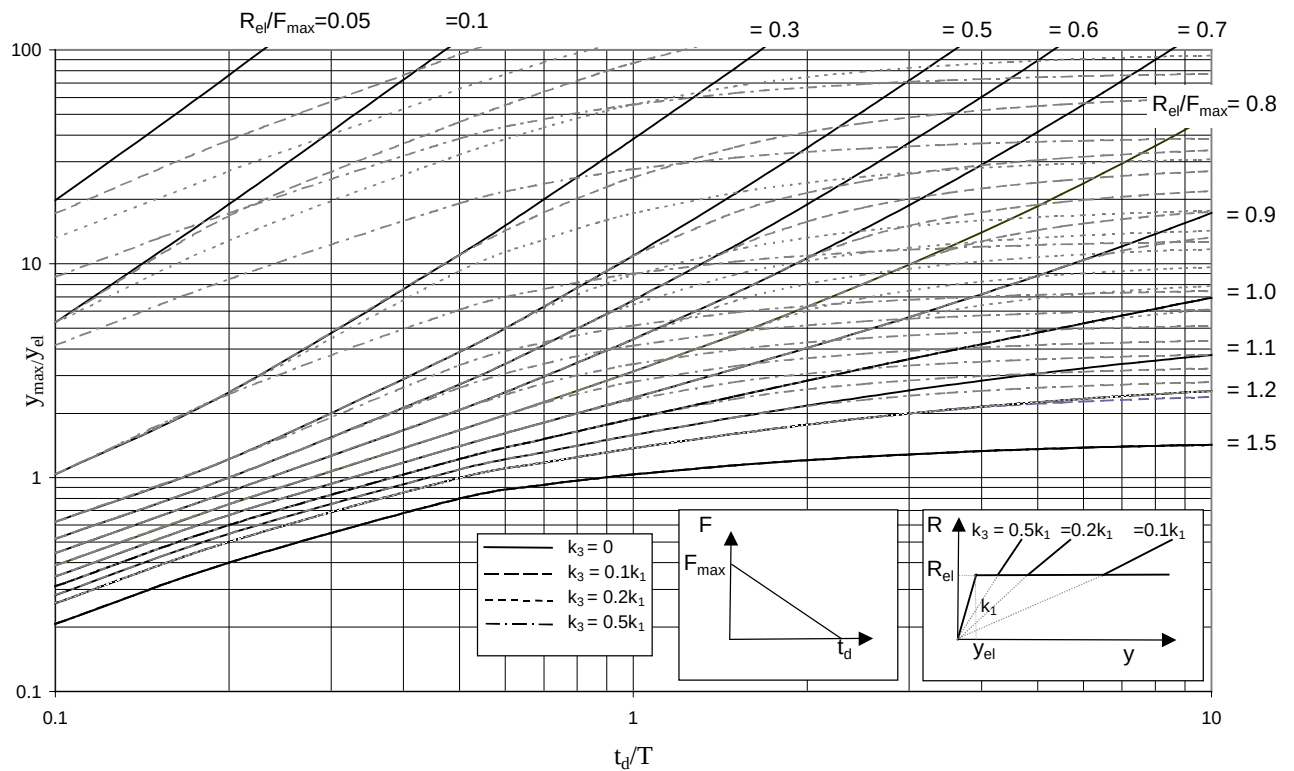


Figure A.6-4 Dynamic response of a SDOF system to a triangular load (rise time=0)

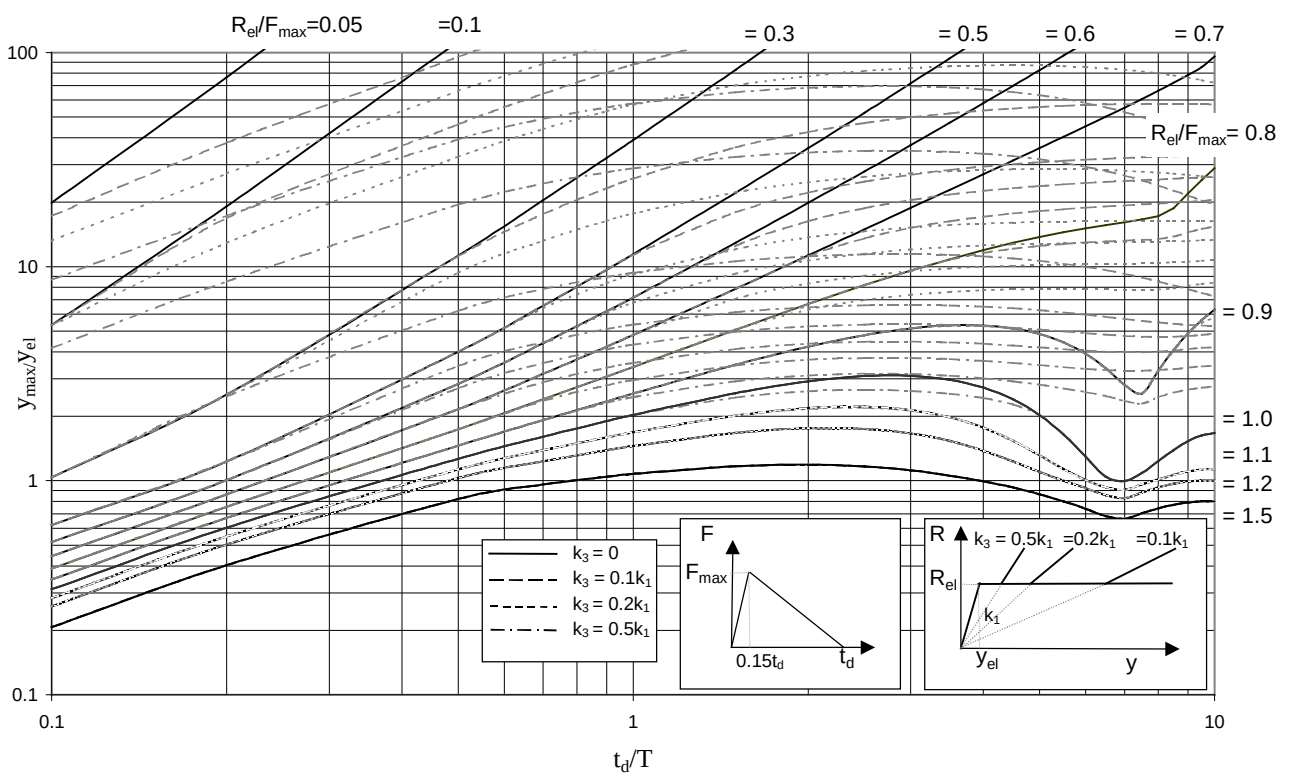


Figure A.6-5 Dynamic response of a SDOF system to a triangular load (rise time=0.15 t_d)

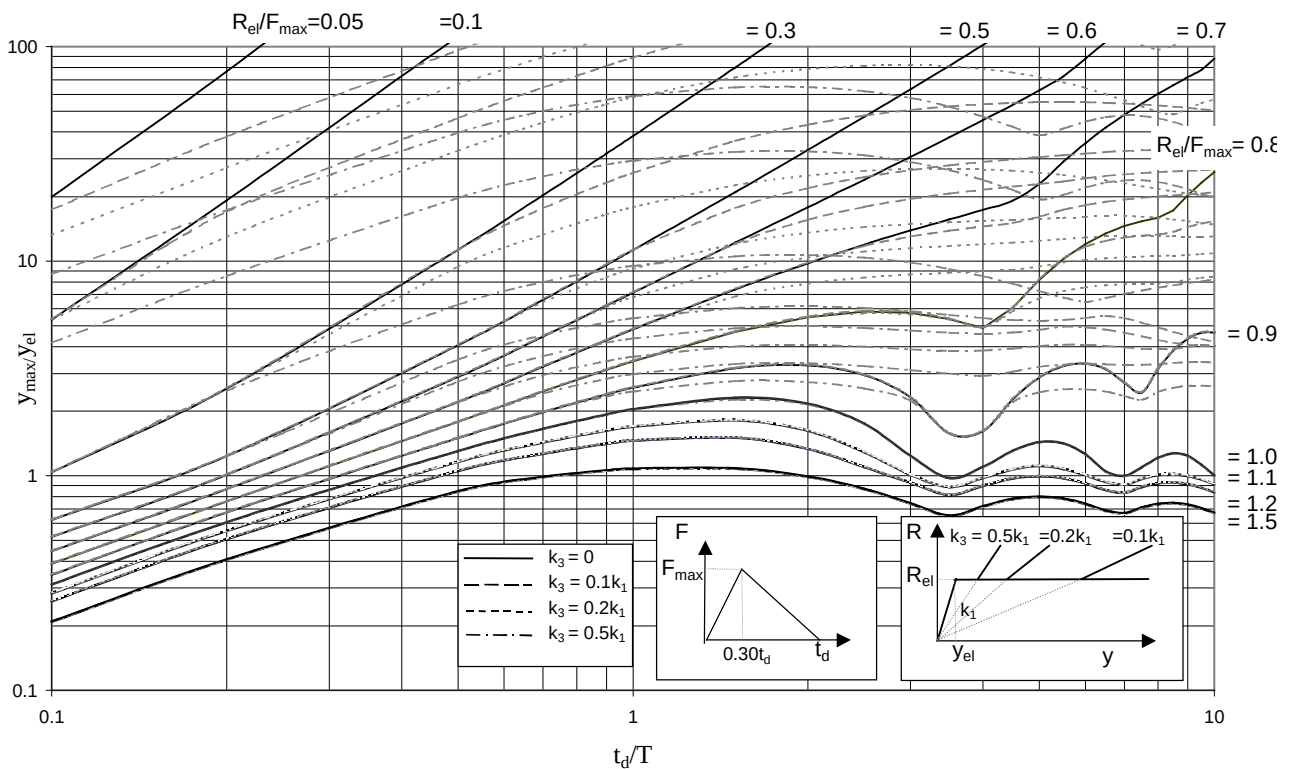


Figure A.6-6 Dynamic response of a SDOF system to a triangular load (rise time=0.30 t_d)

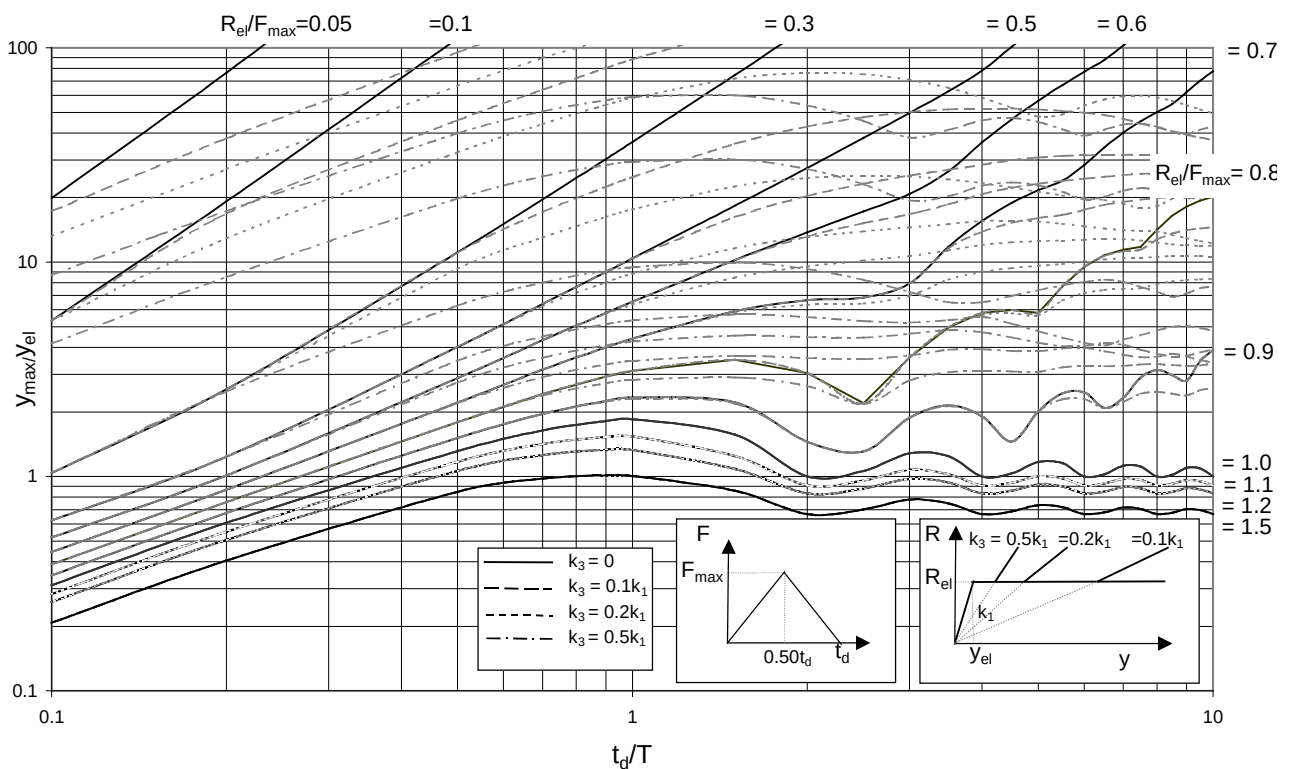


Figure A.6-7 Dynamic response of a SDOF system to a triangular load (rise time=0.50 t_d)

A.6.5 MDOF analysis

SDOF analysis of built-up structures (e.g. stiffeners supported by girders) is admissible if

- the fundamental periods of elastic vibration are sufficiently separated
- the response of the component with the smallest eigenperiod does not enter the elastic-plastic domain so that the period is drastically increased

If these conditions are not met, then significant interaction between the different vibration modes is anticipated and a multi degree of freedom analysis is required with simultaneous time integration of the coupled system.

A.6.6 Classification of resistance properties

A.6.6.1 Cross-sectional behaviour

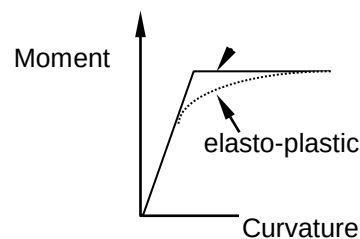


Figure A.6-8 Bending moment-curvature relationships

Elasto-plastic : The effect of partial yielding on bending moment accounted for

Elastic-perfectly plastic: Linear elastic up to fully plastic bending moment

The simple models described herein assume elastic-perfectly plastic material behaviour. Any strain hardening may be accounted for by equivalent (increased) yield stress.

A.6.6.2 Component behaviour

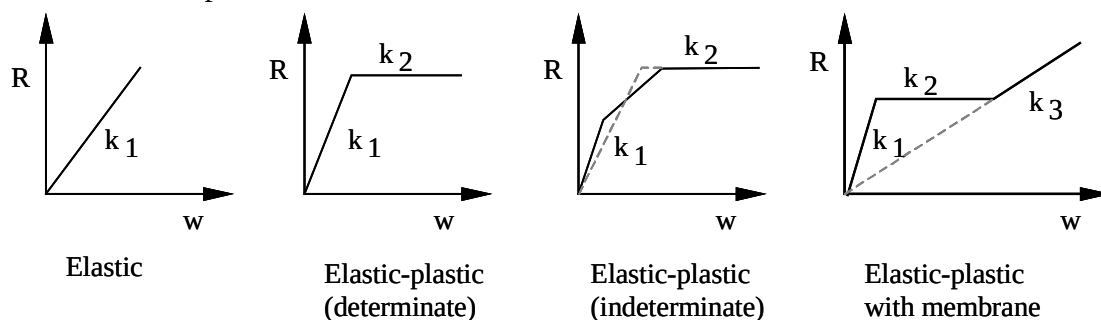


Figure A.6-9 Resistance curves

Elastic: Elastic material, small deformations

Elastic-plastic (determinate): Elastic-perfectly plastic material. Statically determinate system. Bending mechanism fully developed with occurrence of first plastic hinge(s)/yield lines. No axial restraint

Elastic-plastic (indeterminate): Elastic perfectly plastic material. Statically indeterminate system. Bending mechanism develops with sequential formation of plastic hinges/yield lines. No axial restraint. For simplified analysis this system may be modelled as an elastic-plastic (determinate) system with equivalent initial stiffness. In lieu of more accurate analysis the equivalent stiffness should be determined such that the area under the resistance curve is preserved.

Elastic-plastic with membrane: Elastic-perfectly plastic material. Statically determinate (or indeterminate). Ends restrained against axial displacement. Increase in load-carrying capacity caused by development of membrane forces .

A.6.7 Idealisation of resistance curves

The resistance curves in clause A.6.6 are idealised. For simplified SDOF analysis the resistance characteristics of a real non-linear system may be approximately modelled. An example with a tri-linear approximation is illustrated in Figure A.6-10.

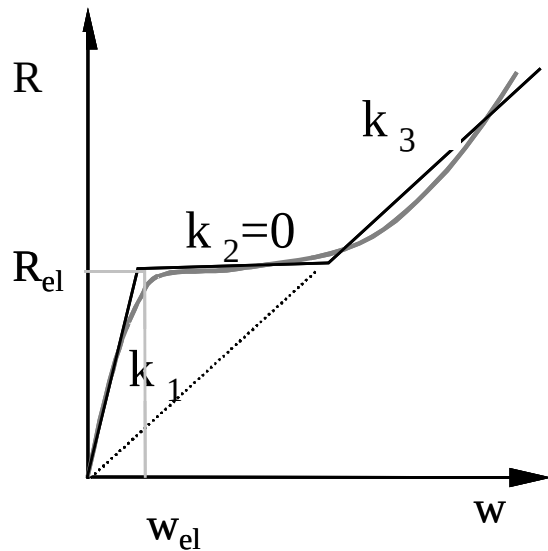


Figure A.6-10 Representation of a non-linear resistance by an equivalent tri-linear system.

In lieu of more accurate analysis the resistance curve of elastic-plastic systems may be composed by an elastic resistance and a rigid-plastic resistance as illustrated in Figure A.6-11.

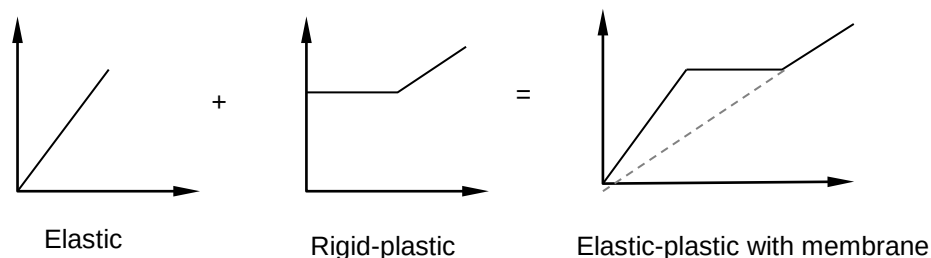


Figure A.6-11 Construction of elastic-plastic resistance curve

A.6.8 Resistance curves and transformation factors for plates

A.6.8.1 Elastic - rigid plastic relationships

In lieu of more accurate calculations rigid plastic theory combined with elastic theory may be used.

In the elastic range the stiffness and fundamental period of vibration of a clamped plate under uniform lateral pressure can be expressed as:

$$r = k_1 w \quad = \quad \text{resistance-displacement relationship for plate center}$$

$$k_1 = \psi \frac{D}{s^4} \quad = \quad \text{plate stiffness}$$

$$T = \frac{2\pi}{\eta} \sqrt{\frac{\rho t s^4}{D}} \quad = \quad \text{natural period of vibration}$$

$$D = E \frac{t^3}{12(1-\nu^2)} = \text{plate bending stiffness}$$

The factors ψ and η are given in Figure A.6-12

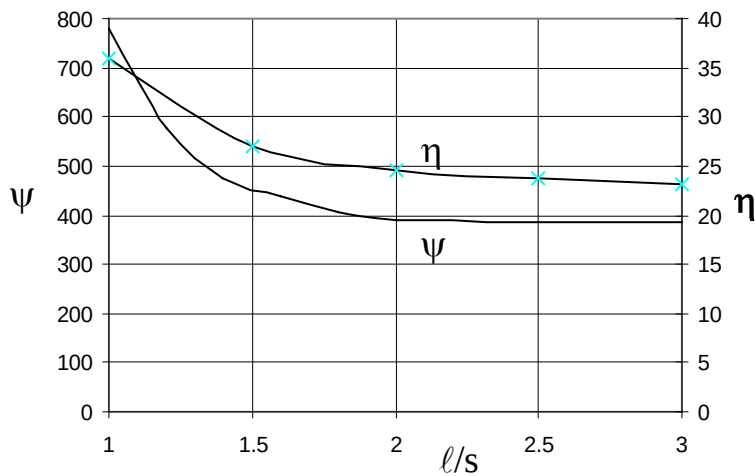


Figure A.6-12 Coefficients ψ and η .

In the plastic range the resistance of plates subjected to uniform pressure can be taken as:

$$\frac{p}{p_c} = 1 + \bar{w}^2 \left(\frac{\alpha + (3-2\alpha)^2}{9-3\alpha} \right) \quad \bar{w} \leq 1 \quad (\text{A.6.8})$$

$$\frac{p}{p_c} = 2\bar{w} \left(1 + \frac{\alpha(2-\alpha)}{3-\alpha} \left(\frac{1}{3\bar{w}^2} - 1 \right) \right) \quad \bar{w} \geq 1$$

Pinned ends :

$$\bar{w} = 2 \frac{w}{t} \quad r_c = \frac{6f_y t^2}{\ell^2 \alpha^2}$$

Clamped ends :

$$\bar{w} = \frac{w}{t} \quad r_c = \frac{12f_y t^2}{\ell^2 \alpha^2}$$

$$\alpha = \frac{s}{\ell} \left(\sqrt{3 + \left(\frac{s}{\ell} \right)^2} - \frac{s}{\ell} \right) = \text{plate aspect parameter}$$

- $\ell (> s)$ = plate length
- s = plate width
- t = plate thickness
- r_c = plastic resistance in bending for plates with no axial restraint

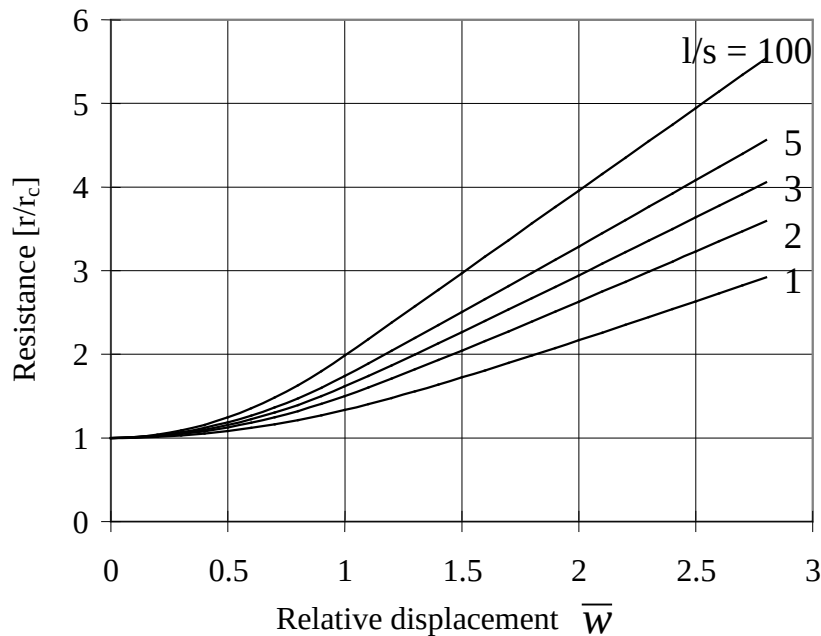


Figure A.6-13 Plastic load-carrying capacities of plates as a function of lateral displacement

A.6.8.2 Axial restraint

Unlike stiffeners no simple method with a clear physical interpretation exists to quantify the effect of flexible boundaries for a continuous plate field under uniform pressure. Full axial restraint may probably be assumed.

At the panel boundaries assumption of full axial restraint is non-conservative. In lieu of more accurate calculation the axial restraint may be estimated by removing the plate and apply a uniformly distributed unit in-plane load normal to the plate edges. The axial stiffness should be taken as the inverse of the maximum in-plane displacement of the long edge. The relative reduction of the plastic load-carrying capacity can be calculated according to the procedure described in Section A.6.9.2 for a beam with rectangular cross-section (plate thickness x unit width) and length equal to stiffener spacing, using the diagram for $\alpha = 2$.

For a plate in the middle of a continuous, uniformly loaded panel a high degree of axial restraint is likely. A non-dimensional spring factor $c = 1.0$ is suggested.

If membrane forces are likely to develop it has to be verified that the adjacent structure is strong enough to anchor fully plastic membrane forces.

A.6.8.3 Tensile fracture of yield hinges

In lieu of more accurate calculations the procedure described in Section A.3.10.5 may be used for a beam with rectangular cross-section (plate thickness x unit width) and length equal to stiffener spacing.

A.6.9 Resistance curves and transformation factors for beams

Provided that the stiffeners/girders remain stable against local buckling, tripping or lateral torsional buckling stiffened plates/girders may be treated as beams. Simple elastic-plastic methods of analysis are generally applicable. Special considerations shall be given to the effect of:

- Elastic flexibility of member/adjacent structure
- Local deformation of cross-section
- Local buckling
- Strength of connections
- Strength of adjacent structure
- Fracture

A.6.9.1 Beams with no- or full axial restraint

Equivalent springs and transformation factors for load and mass for various idealised elasto-plastic systems are shown in Table A.6-2. For more than two concentrated loads, use values for uniform loading.

Shear deformation may have a significant impact on the elastic flexibility and eigenperiod of beams and girders with a short span/web height ratio (L/h_w), notably for clamped ends. The eigenperiod and stiffness in the linear domain including shear deformation may be calculated as:

$$T = 2\pi \sqrt{\frac{m}{k}} = 2\pi \sqrt{\frac{K_{l,m,u} M_u + K_{l,m,c} M_c}{k_1}} \sqrt{1 + \left(c_s \frac{\pi r_g}{L} \right)^2 \left(1 + \frac{E}{G} \frac{A}{A_w} \right)} \quad (\text{A.6.9})$$

and

$$\frac{1}{k_1} = \frac{1}{k_1} + \frac{1}{k_Q}, \quad k_Q = c_Q \frac{GA_w}{L} \quad (\text{A.6.10})$$

where

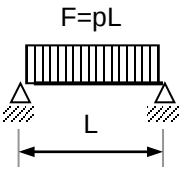
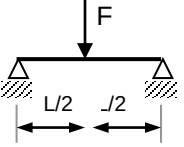
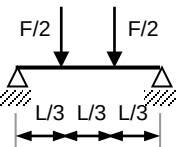
c_s	= 1.0 for both ends simply supported
c_s	= 1.25 for one end clamped and one end simply supported
c_s	= 1.5 for both ends clamped
L	= length of beam/girder
E	= elastic modulus
G	= shear modulus
A	= total cross-sectional area of beam/girder
A_w	= shear area of beam/girder
r_g	= radius of gyration
k_Q	= shear stiffness for beam/girder
k_1	= bending stiffness in the linear domain according to Table A.6-2
k_1	= beam stiffness due to bending and shear
k_e	= equivalent bending in the elasto-plastic domain
M_{ps}	= plastic bending capacity of beam at support
M_{pm}	= plastic bending capacity of beam at midspan

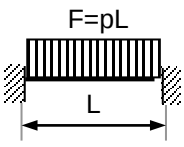
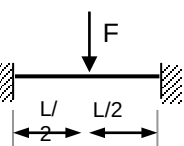
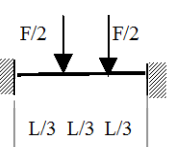
and regardless of rotational boundary conditions the following values may be used

c_Q	= 8 for uniformly distributed loads
c_Q	= 4 for one concentrated loads
c_Q	= 6 for two concentrated loads

The dynamic reactions according to Table A.6-2 become increasingly inaccurate for loads with short duration and/or high magnitudes.

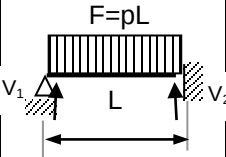
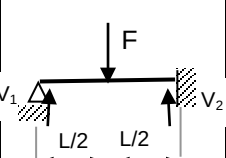
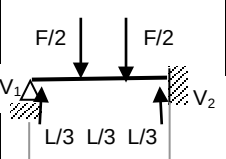
Table A.6-2 Transformation factors for beams with various boundary and load conditions.

Load case	Resistance domain	Load Factor K_l	Mass factor K_m		Load-mass factor K_{lm}		Maximum resistance R_{el}	Linear stiffness k_1	Dynamic reaction V
			Concentrated mass	Uniform mass	Concentrated mass	Uniform mass			
	Elastic	0.64		0.50		0.78	$\frac{8M_p}{L}$	$\frac{384EI}{5L^3}$	$0.39R + 0.11F$
	Plastic bending	0.50		0.33		0.66	$\frac{8M_p}{L}$	0	$0.38R_{el} + 0.12F$
	Plastic membrane	0.50		0.33		0.66		$\frac{4N_p}{L}$	$\frac{2N_p y_{max}}{L}$
	Elastic	1.0	1.0	0.49	1.0	0.49	$\frac{4M_p}{L}$	$\frac{48EI}{L^3}$	$0.78R - 0.28F$
	Plastic bending	1.0	1.0	0.33	1.0	0.33	$\frac{4M_p}{L}$	0	$0.75R_{el} - 0.25F$
	Plastic membrane	1.0	1.0	0.33	1.0	0.33	$\frac{4N_p}{L}$		$\frac{2N_p y_{max}}{L}$
	Elastic	0.87	0.76	0.52	0.87	0.60	$\frac{6M_p}{L}$	$\frac{56.4EI}{L^3}$	$0.525R - 0.025F$
	Plastic bending	1.0	1.0	0.56	1.0	0.56	$\frac{6M_p}{L}$	0	$0.52R_{el} - 0.02F$
	Plastic membrane	1.0	1.0	0.56	1.0	0.56		$\frac{6N_p}{L}$	$\frac{3N_p y_{max}}{L}$

Load case	Resistance domain	Load Factor K_l	Mass factor K_m		Load-mass factor K_{lm}		Maximum resistance R_{el}	Linear stiffness k_1	Equivalent linear stiffness k_e	Dynamic reaction V
			Concentrated mass	Uniform mass	Concentrated mass	Uniform mass				
	Elastic	0.53		0.41		0.77	$\frac{12 M_{ps}}{L}$	$\frac{384EI}{L^3}$	$307 \cdot m_1$	$0.36R + 0.14 F$
	Elasto-plastic bending	0.64		0.50		0.78	$\frac{8(M_{ps} + M_{pm})}{L}$	$\frac{384EI}{5L^3}$		$0.39R_{el} + 0.11F$
	Plastic bending	0.50		0.33		0.66	$\frac{8(M_{ps} + M_{pm})}{L}$	0		$0.38R_{el} + 0.12F$
	Plastic membrane	0.50		0.33		0.66		$\frac{4N_p}{L}$		$\frac{2N_{py_{max}}}{L}$
	Elastic	1.0	1.0	0.37	1.0	0.37	$\frac{4(M_{ps} + M_{pm})}{L}$	$\frac{192EI}{L^3}$	$48 \cdot m_1$	$0.71R - 0.21F$
	Plastic bending	1.0	1.0	0.33	1.0	0.33	$\frac{4(M_{ps} + M_{pm})}{L}$	0		$0.75R_{el} - 0.25F$
	Plastic membrane	1.0	1.0	0.33	1.0	0.33		$\frac{4N_p}{L}$		$\frac{2N_{py_{max}}}{L}$
	Elastic	0.80	0.64	0.41	0.80	0.51	$\frac{9M_{ps}}{L}$	$\frac{260EI}{L^3}$	$212 \cdot m_1$	$0.48R + 0.02F$
	Elasto-plastic bending	0.87	0.76	0.52	0.87	0.60	$\frac{6(M_{ps} + M_{pm})}{L}$	$\frac{56.4EI}{L^3}$		$0.52R_{el} - 0.02F$
	Plastic bending	1.0	1.0	0.56	1.0	0.56	$\frac{6(M_{ps} + M_{pm})}{L}$	0		$0.52R_{el} - 0.02F$
	Plastic membrane	1.0	1.0	0.56	1.0	0.56		$\frac{6N_p}{L}$		$\frac{3N_{py_{max}}}{L}$

q = explosion load per unit length ($q = p_s$ for stiffeners, $q = p_\ell$ for girders)

$$m_1 = \frac{EI}{l^3} \left(\frac{1.5M_{ps}}{M_{ps} + 2M_{pm}} + 0.25 \right)$$

Load case	Resistance domain	Load Factor K_l	Mass factor K_m		Load-mass factor K_{lm}		Maximum resistance R_{el}	Linear stiffness k_1	Equivalent linear stiffness k_1	Dynamic reaction V
			Concentrated mass	Uniform mass	Concentrated mass	Uniform mass				
	Elastic	0.58		0.45		0.78	$\frac{8M_{ps}}{L}$	$\frac{185EI}{L^3}$	160·m ₂	$V_1 = 0.26R + 0.12F$ $V_2 = 0.43R + 0.19F$
	Elasto-plastic bending	0.64		0.50		0.78	$\frac{4(M_{ps} + 2M_{pm})}{L}$	$\frac{384EI}{5L^3}$		$0.39R + 0.11F \pm M_{ps}/L$
	Plastic bending	0.50		0.33		0.66	$\frac{4(M_{ps} + 2M_{pm})}{L}$	0		$0.38R + 0.12F \pm M_{ps}/L$
	Plastic membrane	0.50	0.33		0.66			$\frac{4N_p}{L}$		$\frac{2N_p y_{max}}{L}$
	Elastic	1.0	1.0	0.43	1.0	0.43	$\frac{16M_{ps}}{3L}$	$\frac{107EI}{L^3}$	106·m ₂	$V_1 = 0.25R + 0.07F$ $V_2 = 0.54R + 0.14F$
	Elasto-plastic bending	1.0	1.0	0.49	1.0	0.49	$\frac{2(M_{ps} + 2M_{pm})}{L}$	$\frac{48EI}{L^3}$		$0.78R - 0.28F \pm M_{ps}/L$
	Plastic bending	1.0	1.0	0.33	1.0	0.33	$\frac{2(M_{ps} + 2M_{pm})}{L}$	0		$0.75R - 0.25F \pm M_{ps}/L$
	Plastic membrane	1.0	1.0	0.33	1.0	0.33		$\frac{4N_p}{L}$		$\frac{2N_p y_{max}}{L}$
	Elastic	0.81	0.67	0.45	0.83	0.55	$\frac{6M_{ps}}{L}$	$\frac{132EI}{L^3}$	122·m ₃	$V_1 = 0.17R + 0.17F$ $V_2 = 0.33R + 0.33F$
	Elasto-plastic bending	0.87	0.76	0.52	0.87	0.60	$\frac{2(M_{ps} + 3M_{pm})}{L}$	$\frac{56EI}{L^3}$		$0.525R - 0.025F \pm M_{ps}/L$
	Plastic bending	1.0	1.0	0.56	1.0	0.56	$\frac{2(M_{ps} + 3M_{pm})}{L}$	0		$0.52R_{el} - 0.02F \pm M_{ps}/L$
	Plastic membrane	1.0	1.0	0.56	1.0	0.56		$\frac{6N_p}{L}$		$\frac{3N_p y_{max}}{L}$

$$m_2 = \frac{EI}{l^3} \left(\frac{1.5M_{ps}}{M_{ps} + 2M_{pm}} + 0.5 \right)$$

$$m_3 = \frac{EI}{l^3} \left(\frac{2M_{ps}}{M_{ps} + 3M_{pm}} + 0.5 \right)$$

A.6.9.2 Beams with partial end restraint.

Relatively small axial displacements have a significant influence on the development of tensile forces in members undergoing large lateral deformations. Equivalent elastic, axial stiffness may be defined as

$$\frac{1}{k} = \frac{1}{k_{\text{node}}} + \frac{\ell}{2EA} \quad (\text{A.6.11})$$

k_{node} = axial stiffness of the node with the considered member removed. This may be determined by introducing unit loads in member axis direction at the end nodes with the member removed.

Plastic force-deformation relationship for a beam under uniform pressure may be obtained from Figure A.6-14, Figure A.6-15 or Figure A.6-16 if the plastic interaction between axial force and bending moment can be approximated by the following equation:

$$\frac{M}{M_p} + \left(\frac{N}{N_p} \right)^\alpha = 1 \quad 1 < \alpha < 2 \quad (\text{A.6.12})$$

In lieu of more accurate analysis $\alpha = 1.2$ can be assumed for stiffened plates and H or I beams. For members with tubular section $\alpha = 1.75$.

$R_0 = \frac{8c_1 f_y W_p}{\ell}$	=	plastic collapse resistance in bending for the member.
ℓ	=	member length
$\bar{w} = \frac{w}{c_1 w_c}$	=	non-dimensional deformation
$w_c = \frac{\alpha W_p}{A}$	=	characteristic beam height for beams described by plastic interaction equation (A.6.12).
$c = \frac{4c_1 k w_c^2}{f_y A \ell}$	=	non-dimensional spring stiffness
c_1	=	2 for clamped beams
c_1	=	1 for pinned beams
W_p	=	Plastic section modulus for the cross section of the beam
$W_p = z_g A_s$	=	plastic section modulus for stiffened plate for $s_e t > A_s$
$A = A_s + st$	=	total area of stiffener and plate flange
$A_e = A_s + s_e t$	=	effective cross-sectional area of stiffener and plate flange,
z_g	=	distance from plate flange to stiffener centre of gravity.
A_s	=	stiffener area
s	=	stiffener spacing
s_e	=	effective width of plate flange see A.6.9.3

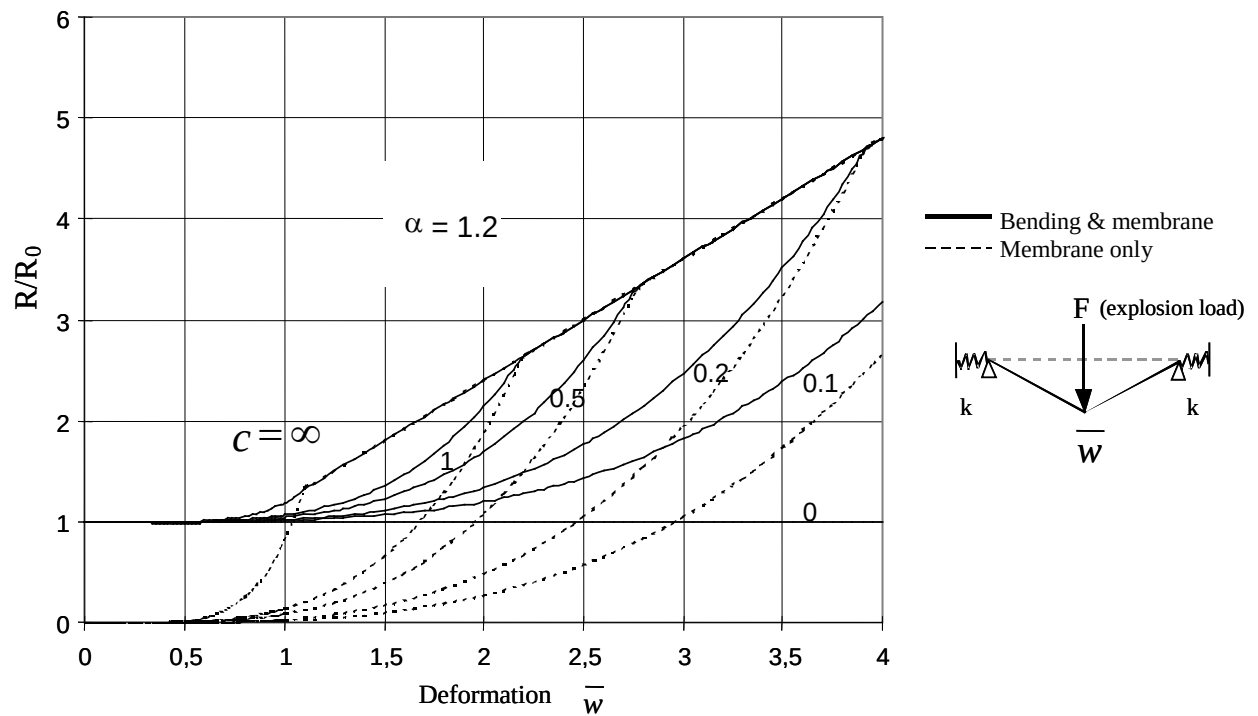


Figure A.6-14 Plastic load-deformation relationship for beam with axial flexibility ($\alpha=1.2$)

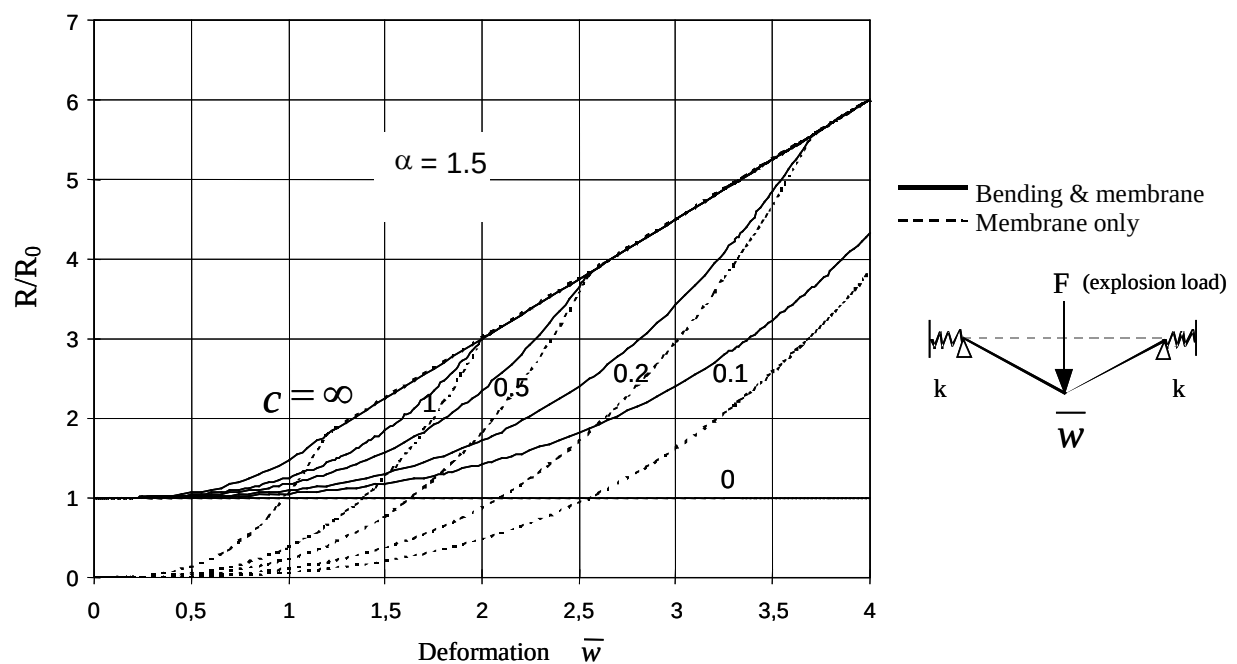


Figure A.6-15 Plastic load-deformation relationship for beam with axial flexibility ($\alpha=1.5$)

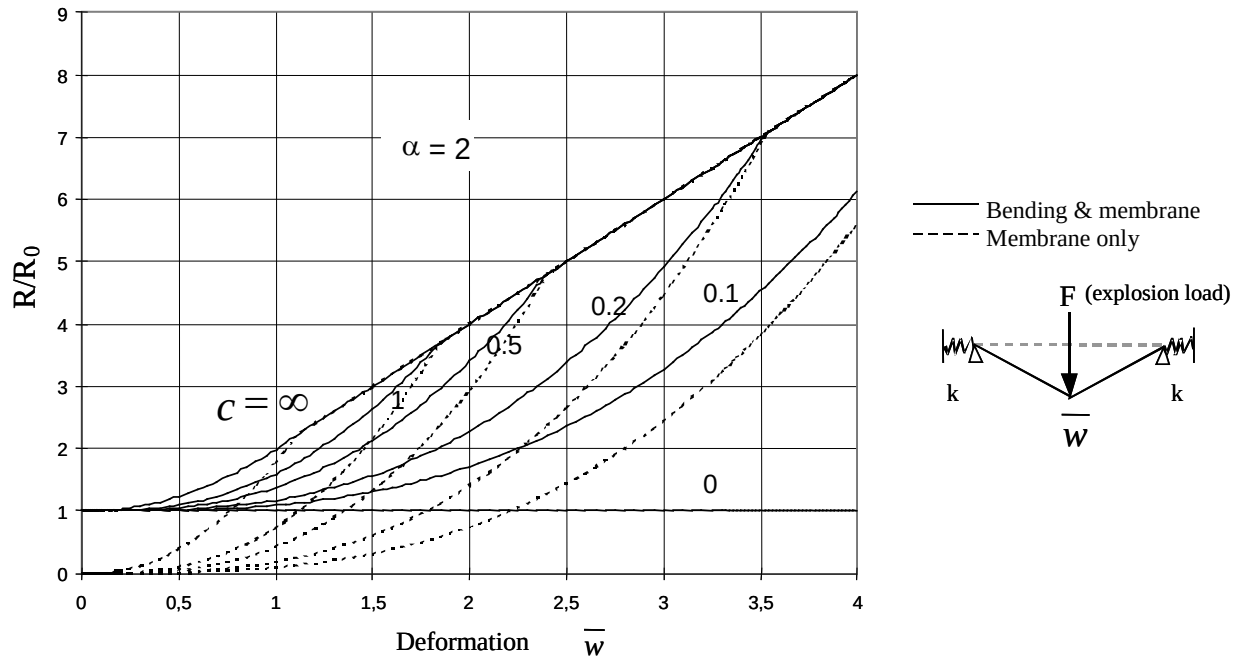


Figure A.6-16 Plastic load-deformation relationship for beam with axial flexibility ($\alpha=2$)

For members where the plastic moment capacity of adjacent members is smaller than the moment capacity of the exposed member the force-deformation relationship may be interpolated from the curves for pinned ends and clamped ends:

$$R = \xi R_{\text{clamped}} + (1 - \xi) R_{\text{pinned}} \quad (\text{A.6.13})$$

where

$$0 \leq \xi = \frac{R_0^{\text{actual}}}{8 \frac{M_p}{\ell}} - 1 \leq 1 \quad (\text{A.6.14})$$

R_0^{actual} = Collapse load in bending for beam accounting for actual bending resistance of adjacent members

$$R_{\text{actual}} = \frac{8M_p + 4M_{p1} + 4M_{p2}}{\ell} \quad (\text{A.6.15})$$

$$M_{pj} = \sum_i M_{pj,i} \leq M_p \quad ; i = \text{adjacent member no } i, \quad j = \text{end number } \{1,2\} \quad (\text{A.6.16})$$

$M_{pj,i}$ = Plastic bending moment for member no. i.

Elastic, rotational flexibility of the node is normally of moderate significance

A.6.9.3 Effective flange

In order to analyse stiffened plate as a beam the effective width of the plate between stiffener need to be determined. The effective width needs to be reduced due to buckling and/or shear lag.

Shear lag effects may be neglected if the length is more than 2.5 times the width between stiffeners. For guidance see Commentary.

Determination of effective flange due to buckling can be made as for buckling of stiffened plates see DNV-RP-C201.

The effective width for elastic deformations may be used when the plate flange is on the tension side.

In most cases the flange will experience varying stress with parts in compression and parts in tension. It may be unduly conservative to use the effective width for the section with the largest compression stress to be valid for the whole member length. For continuous stiffeners it will be reasonable to use the mean value between effective width at the section with the largest compression stress and the full width. For simple supported stiffeners with compression in the plate it is judged to be reasonable to use the effective width at midspan for the total length of the stiffener.

A.6.9.4 Strength of adjacent structure

The adjacent structure must be checked to see whether it can provide the support required by the assumed collapse mechanism for the member/sub-structure

A.6.9.5 Strength of connections

The capacity of connections can be taken from recognised codes

The connection shall be checked for the dynamic reaction force given in Table A.6-2.

For beams with axial restraint the connection should also be checked for lateral - and axial reaction in the membrane phase:

If the axial force in a tension member exceeds the axial capacity of the connection the member must be assumed disconnected.

If the capacity of the connection is exceeded in compression and bending, this does not necessarily mean failure of the member. The post-collapse strength of the connection may be taken into account provided that such information is available.

A.6.9.6 Ductility limits

Reference is made to Section A.3.10

The local buckling criterion in Section A.3.10.2 and tensile fracture criterion given in Section A.3.10.4 may be used with:

- d_c = characteristic dimension equal to twice the distance from the plastic neutral axis in bending to the extreme fibre of the cross-section
- c = non-dimensional axial spring stiffness calculated in Section A.6.9.2

Alternatively, the ductility ratios $\mu = \frac{y_{\max}}{y_{el}}$ in Table A.6-3 may be used.

Table A.6-3 Ductility ratios μ – beams with no axial restraint

Boundary conditions	Load	Cross-section category		
		Class 1	Class 2	Class 3
Cantilevered	Concentrated	6	4	2
	Distributed	7	5	2
Pinned	Concentrated	6	4	2
	Distributed	12	8	3
Fixed	Concentrated	6	4	2
	Distributed	4	3	2

A.7 RESIDUAL STRENGTH

A.7.1 General

The second step in the accidental limit state check is to verify the residual strength of the installation with damage caused by the accidental load.

The check shall be carried out for functional loads and design environmental loads.

The partial safety factor for loads and material can be taken equal to unity.

After the action of the accidental load an internal, residual stress/force field remains in the structure. The resultant of this field is zero. In most cases the residual stresses/forces have a minor influence on the residual capacity and may be neglected. Any detrimental effect of the residual stress on the ultimate capacity should, however, be subject to explicit consideration. If necessary, the residual stresses should be included in the analysis.

If non-linear FE analysis is used, the residual stress field can be conveniently included by performing integrated analysis:

- 1) Application of design accidental loading
- 2) Removal of accidental load by unloading
- 3) Application of design environmental load

A.7.2 Modelling of damaged members

A.7.2.1 General

Compressive members with large lateral deformations will often contribute little to load-carrying and can be omitted from analysis.

A.7.2.2 Members with dents, holes, out-of-straightness

Tubular members with dents, holes and out-of-straightness, reference is made to Chapter 10.

Stiffened plates/Beams/Girders with deformed plate flange and out-of-straightness, reference is made to Chapter 10.

A.8 REFERENCES

- /1/ NORSOK Standard N-003 Action and Action Effect
- /2/ NS-EN 19931-2 Eurocode 3: Design of Steel structures Part 1-2. General rules - Structural fire design
- /3/ Amdahl, J.: "Energy Absorption in Ship-Platform Impacts", UR-83-34, Dept. Marine Structures, Norwegian Institute of Technology, Trondheim, 1983.
- /4/ SCI 1993: Interim Guidance Notes for the Design and Protection of Topside Structures against Explosion and Fire
- /5/ Amdahl, J.: "Mechanics of Ship-Ship Collisions- Basic Crushing Mechanics".West Europe Graduate School of Marine Technology, WEGEMT , Copenhagen, 1995

A.9 COMMENTARY

Comm. A.3.1 General

For typical installations, the contribution to energy dissipation from elastic deformation of component/substructures in direct contact with the ship is very small and can normally be neglected. Consequently, plastic methods of analysis apply.

However, elastic elongation of the hit member as well as axial flexibility of the nodes to which the member is connected, have a significant impact on the development of membrane forces in the member. This effect has to be taken into account in the analysis, which is otherwise based on plastic methods. The diagrams in Section A.3.7.2 are based on such an approach.

Depending on the structure size/configuration as well as the location of impact elastic strain energy over the entire structure may contribute significantly.

Comm. A.3.2 Design principles

The transition from essentially strength behaviour to ductile response can be very abrupt and sensitive to minor changes in scantlings. E.g. integrated analyses of impact between the stern of a supply vessel and a large diameter column have shown that with moderate change of (ring - and longitudinal) stiffener size and/or spacing, the energy dissipation may shift from predominantly platform based to predominantly vessel based. Due attention should be paid to this sensitivity when the calculation procedure described in Section A.3.5 is applied.

Comm. A.3.5.1 Recommended force-deformation relationship

The curve for bow impact in Figure A.3-4 has been derived on the assumption of impacts against an infinitely rigid wall. Sometimes the curve has been used erroneously to assess the energy dissipation in bow-brace impacts.

Experience from small-scale tests /3/ indicates that the bow undergoes very little deformation until the brace becomes strong enough to crush the bow. Hence, the brace absorbs most of the energy. When the brace is strong enough to crush the bow the situation is reversed; the brace remains virtually undamaged.

On the basis of the tests results and simple plastic methods of analysis, force-deformation curves for bows subjected to (strong) brace impact were established in /3/ as a function of impact location and brace diameter. These curves are reproduced in Figure A.9-1. In order to fulfil a strength design requirement the brace should at least resist the load level indicated by the broken line (recommended design curve). For braces with a diameter to thickness ratio < 40 it should be sufficient to verify that the plastic collapse load in bending for the brace is larger than the required level. For larger diameter to thickness ratios local denting must probably be taken into account.

Normally sized jacket braces are not strong enough to resist the likely bow forces given in Figure A.9-1, and therefore it has to be assumed to absorb the entire strain energy. For the same reasons it has also to be assumed that the brace has to absorb all energy for stern and beam impact with supply vessels.

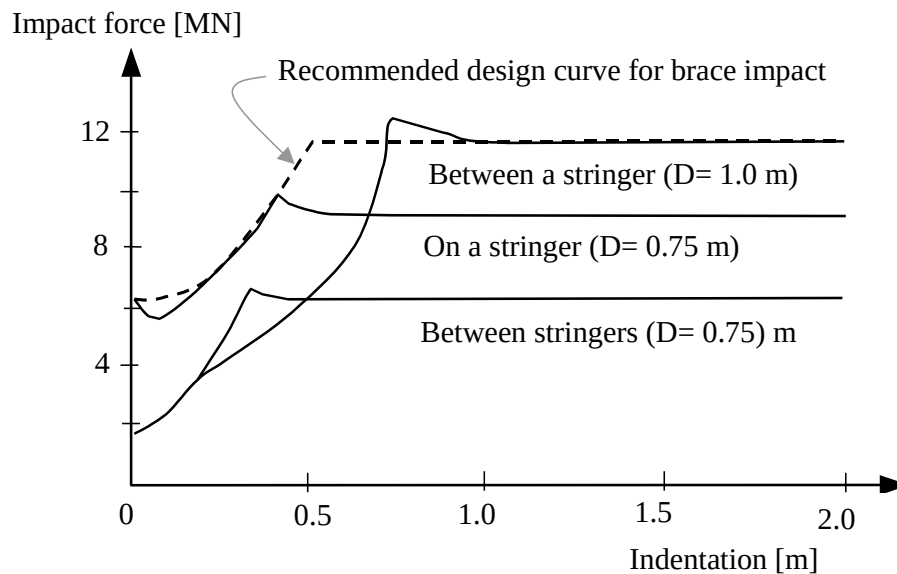


Figure A.9-1 Load-deformation curves for bow-bracing impact /3/

Comm. A.3.5.2 Force contact area for strength design of large diameter columns.

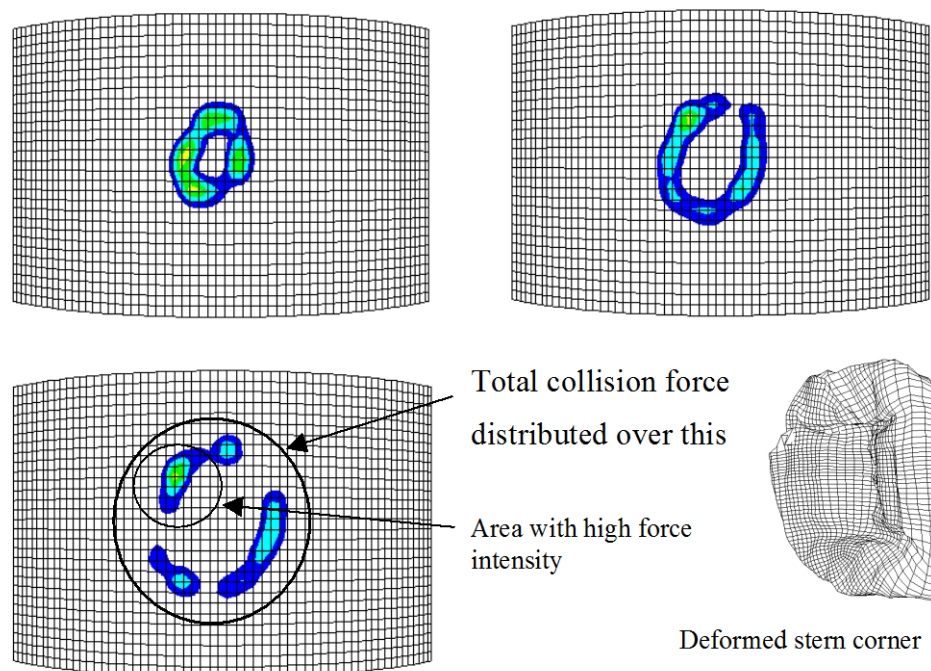


Figure A.9-2 Distribution of contact force for stern corner/large diameter column impact

Figure A.9-2 shows an example of the evolution of contact force intensity during a collision between the stern corner of a supply vessel and a stiffened column. In the beginning the contact is concentrated at the extreme end of the corner, but as the corner deforms it undergoes inversion and the contact ceases in the central part. The contact area is then, roughly speaking, bounded by two concentric circles, but the distribution is uneven.

The force-deformation curves given in Figure A.3-4 relate to total collision force for stern end - and stern corner impact, respectively. Table A.3-1 and Table A.3-2 give the anticipated maximum force intensities (local force and local contact areas, i.e. subsets of the total force and total area) at various stages of deformation.

The basis for the design curves is integrated, non-linear finite element analysis of stern/column impacts.

The information given in A.3.5.2 may be used to perform strength design. If strength design is not achieved numerical analyses have shown that the column is likely to undergo severe deformations and absorb a major part of the strain energy. In lieu of more accurate calculations (e.g. non-linear FEM) it has to be assumed that the column absorbs all strain energy.

Comm. A.3.5.3 Energy dissipation in ship bow.

The requirements in this paragraph are based upon considerations of the relative resistance of a tubular brace to local denting and the bow to penetration of a tubular beam. A fundamental requirement for penetration of the brace into the bow is, first - the brace has sufficient resistance in bending, second - the cross-section does not undergo substantial local deformation. If the brace is subjected to local denting, i.e. undergoes flattening, the contact area with the bow increases and the bow inevitably gets increased resistance to indentation. The provisions ensure that both requirements are complied with.

shows the minimum thickness as a function of brace diameter and resistance level in order to achieve sufficient resistance to penetrate the ship bow without local denting. It may seem strange that the required thickness becomes smaller for increasing diameter, but the brace strength, globally as well as locally, decreases with decreasing diameter.

Local denting in the bending phase can be disregarded provided that the following relationship holds true:

$$\frac{D}{t} \leq 0.14 \frac{1}{C_1^2} \left(\frac{\ell}{D} \right)^2 \quad (\text{A.9.1})$$

Figure A.9-4 shows brace thickness as a function of diameter and length diameter ratio that results from Equation (A.9.1). The thickness can generally be smaller than the values shown, and still energy dissipation in the bow may be taken into account, but if Equation (A.9.1) is complied with denting does not need to be further considered.

The requirements are based upon simulation with LS-DYNA for penetration of a tube with diameter 1.0 m. Great caution should therefore be exercised in extrapolation to diameters substantially larger than 1.0 m, because the resistance of the bow will increase. For brace diameters smaller than 1.0 m, the requirement is conservative.

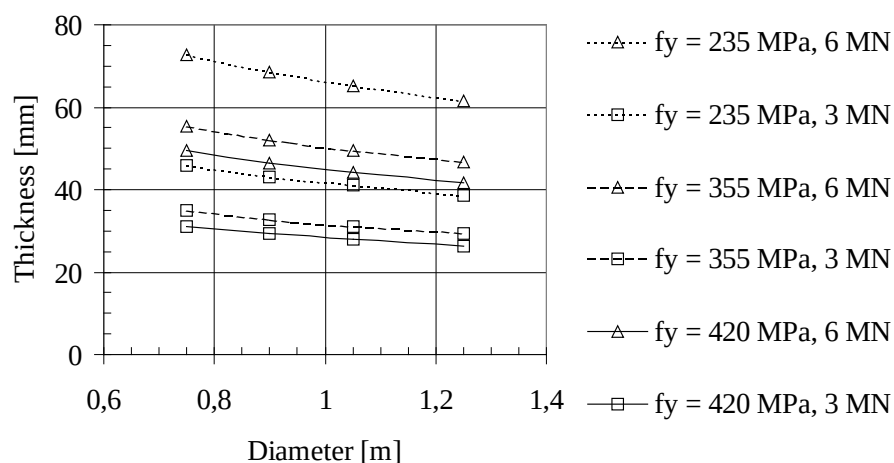


Figure A.9-3 Required thickness versus grade and resistance level of brace to penetrate ship bow without local denting

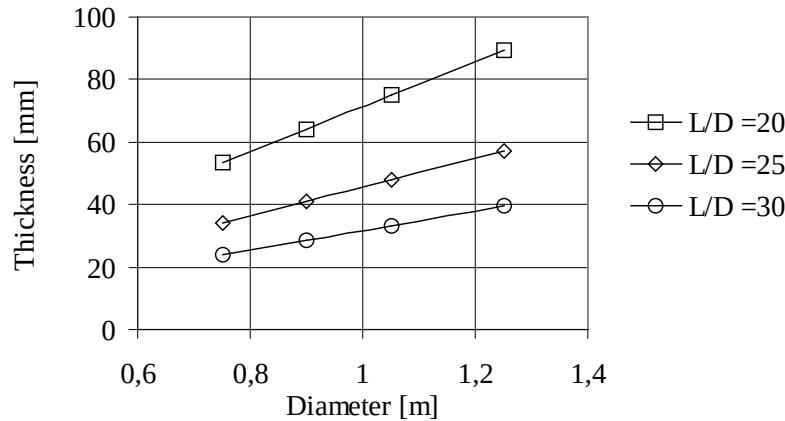


Figure A.9-4 Brace thickness yielding little local deformation in the bending phase

Comm. A.3.10.1 General

If the degradation of bending capacity of the beam cross-section after buckling is known the load-carrying capacity may be interpolated from the curves with full bending capacity and no bending capacity according to the expression:

$$R(\bar{w}) = R_{M_p=1}(\bar{w})\xi + R_{M_p=0}(\bar{w})(1 - \xi) \quad (\text{A.9.2})$$

$R_{M_p=1}(\bar{w})$ = Collapse load with full bending contribution

$R_{M_p=0}(\bar{w})$ = Collapse load with no bending contribution

$$\xi = \frac{R_{M_p,red}}{R_{M_p=1}(\bar{w} = 0)}$$

$R_{M_p,red}$ = Plastic collapse load in bending with reduced cross-sectional capacities. This has to be updated along with the degradation of cross-sectional bending capacity.

Comm. A.3.10.5 Tensile fracture in yield hinges

The rupture criterion is calculated using conventional beam theory. A linear strain hardening model is adopted. For a cantilever beam subjected to a concentrated load at the end, the strain distribution along the beam can be determined from the bending moment variation. In Figure A.9-5 the strain variation,

$\varepsilon = \varepsilon_{cr} / \varepsilon_Y$, is shown for a circular cross-section for a given hardening parameter. The extreme importance of strain hardening is evident; with no strain hardening the high strains are very localised close to the support ($x = 0$), with strain hardening the plastic zone expands dramatically.

On the basis of the strain distribution the rotation in the plastic zone and the corresponding lateral deformation can be determined.

If the beam response is affected by development of membrane forces it is assumed that the membrane strain follows the same relative distribution as the bending strain. By introducing the kinematic relationships for beam elongation, the maximum membrane strain can be calculated for a given displacement.

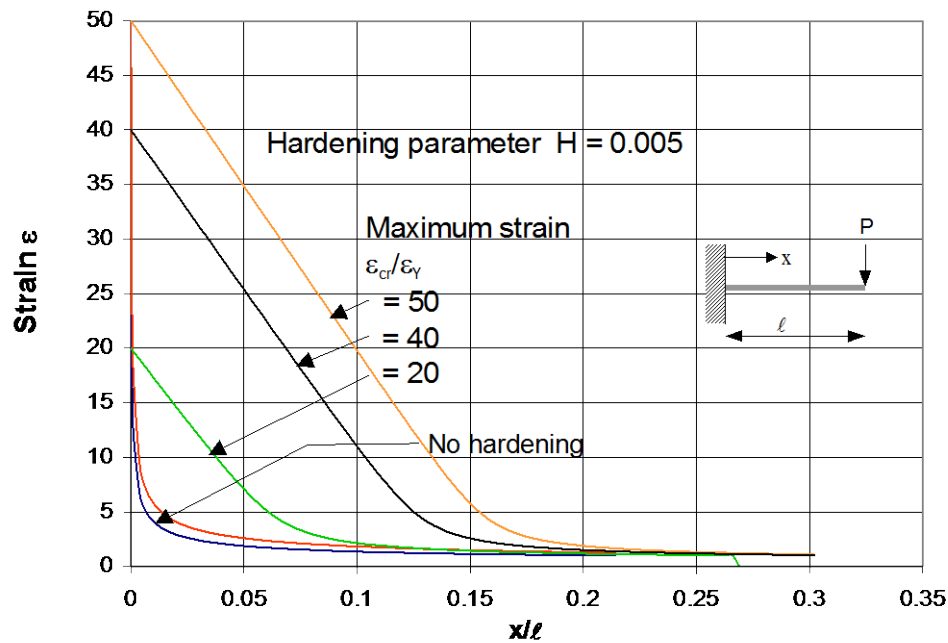


Figure A.9-5 Axial variation of maximum strain for a cantilever beam with circular cross-section

Adding the bending strain and the membrane strain allows determination of the critical displacement as a function of the total critical strain.

Figure A.9-6 shows deformation at rupture for a fully clamped beam as a function of the axial flexibility factor c .

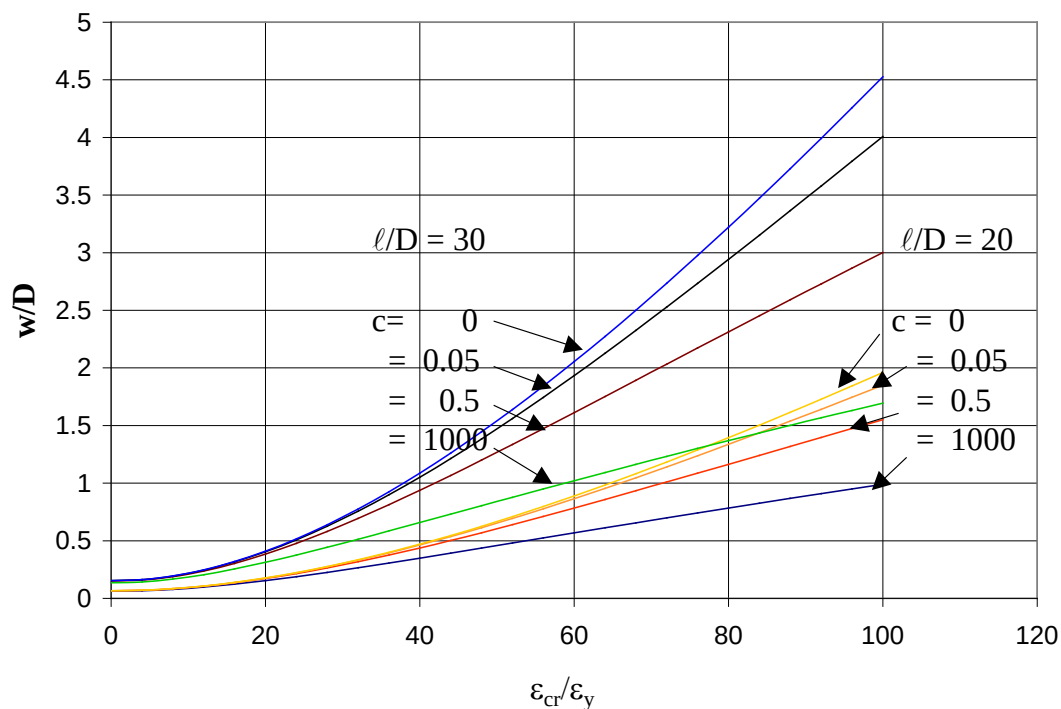


Figure A.9-6 Maximum deformation for a tubular fully clamped beam. ($H=0.005$)

The plastic stiffness factor H is determined from the stress-strain relationship for the material. The equivalent linear stiffness shall be determined such that the total area under the stress-strain curve up to the critical strain is preserved (The two portions of the shaded area shall be equal), refer Figure A.9-7. It is unconservative and not allowable to use a reduced effective yield stress and a plastic stiffness factor as illustrated in Figure A.9-8.

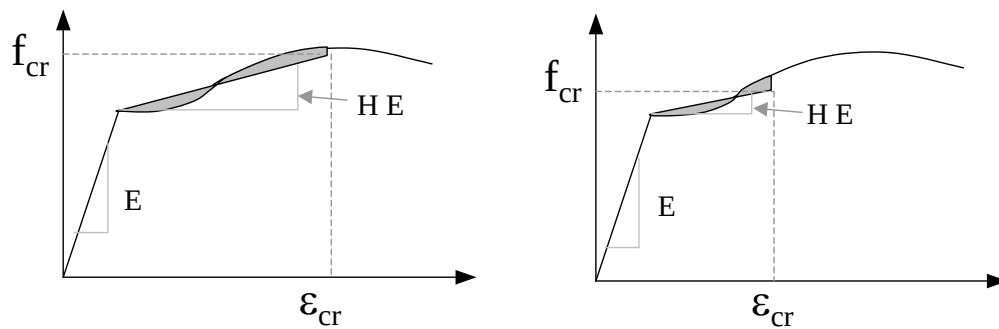


Figure A.9-7 Determination of plastic stiffness

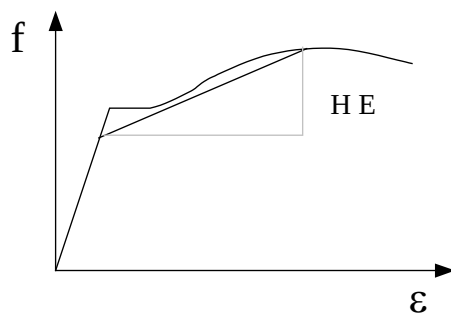


Figure A.9-8 Erroneous determination of plastic stiffness

Comm. A.4.4.1 Stiffened plates subjected to drill collar impact

The validity for the energy expression A.4.6 is limited to $7 < 2r/d < 41$, $t/d < 0.22$ and $m_i/m < 0.75$.

The formula neglect the local energy dissipation which can be added as $E_{loc} = R \cdot 0.2 t$.

In case of hit near the plate edges the energy dissipation will be low and may lead to unreasonable plate thickness. The failure criterion used for the formula is locking of the plate. In many cases locking may be acceptable as long as the falling object is stopped. If the design is based on a hit in the central part of a plate with use of the smaller diameter in the treaded part in the calculations, no penetration of the drill collar will take place at any other hit location due to the collar of such dropped objects.

Comm. A.5.1 General

For redundant structures thermal expansion may cause buckling of members below 400°C . Forces due to thermal expansion are, however, purely internal and will be released once the member buckles. The net effect of thermal expansion is therefore often to create lateral distortions in heated members. In most cases these lateral distortions will have a moderate influence on the ultimate strength of the system.

As thermal expansion is the source of considerable inconvenience in conjunction with numerical analysis it would tempting to replace its effect by equivalent, initial lateral member distortions. There is however, not sufficient information to support such a procedure at present.

Comm. A.5.5 Empirical correction factor

In Eurocode 3 an empirical reduction factor of 1.2 is applied in order to obtain better fit between test results and column curve c for fire exposed compressive members. In the design check this is performed by multiplying the design axial load by 1.2. In non-linear analysis such a procedure is, impractical. In non-linear space frame, stress resultants based analysis the correction factor can be included by dividing the yield compressive load and the Euler buckling load by a factor of 1.2. (The influence of axial force on members stiffness is accounted for by the so-called Livesly's stability multipliers, which are functions of the Euler buckling load.) In this way the reduction factor is applied consistently to both elastic and elasto-plastic buckling.

The above correction factor comes in addition to the reduction caused by yield stress and elastic modulus degradation at elevated temperature if the reduced slenderness is' larger than 0.2.

Comm. A.6.2 Classification of response

Equation (A.6.2) is derived using the principle of conservation of momentum to determine the kinetic energy of the component at the end of the explosion pulse. The entire kinetic energy is then assumed dissipated as strain energy.

Equation (A.6.3) is based on the assumption that the explosion pressure has remained at its peak value during the entire deformation and equates the external work with the total strain energy. In general, the explosion pressure is not balanced by resistance, giving rise to inertia forces. Eventually, these inertia forces will be dissipated as strain energy.

Equation (A.6.4) is based on the assumption that the pressure increases slowly so that the static condition (pressure balanced by resistance) applies during the entire deformation.

Comm. A.6.4 SDOF system analogy

The displacement at the end of the initial, linear resistance domain y_{el} will generally *not* coincide with the displacement at first yield. Typically, y_{el} represents the displacement at the initiation of a plastic collapse mechanism. Hence, y_{el} is larger than the displacement at first yield for two reasons:

- i. Change from elastic to plastic stress distribution over beam cross-section
- ii. Bending moment redistribution over the beam (redundant beams) as plastic hinges form

Comm. A.6.4.2 Dynamic response charts for SDOF system

Figure A.6-3 is derived from the dynamic response chart for a SDOF system subjected to a triangular load with zero rise time given in Figure A.6-4.

In the example it is assumed that from ductility considerations for the assumed mode of deformation a maximum displacement of ten times elastic limit is acceptable. Hence the line

$\frac{y_{allow}}{y_{el}} = \frac{y_{max}}{y_{el}} = 10$ represents the upper limit for the displacement of the component. From the diagram it is

seen that several combinations of pulses characterised by F_{max} and t_d may produce this displacement limit. Each intersection with a response curve (e.g. $k_3 = 0$) yields a normalized pressure

$$\frac{F}{R} = \frac{F_{max}}{R_{el}}$$

and a normalised impulse

$$\frac{I}{RT} = \frac{\frac{1}{2} F_{max} t_d}{R_{el} T} = \frac{1}{2 \frac{R_{el}}{F_{max}}} \cdot \frac{t_d}{T}$$

By plotting corresponding values of normalised impulse and normalised pressure the iso-damage curve given in Figure A.6-3 is obtained.

Comm. A.6.9.3 Effective flange

For deformations in the elastic range the effective width (shear lag effect) of the plate flange, s_e , of simply supported or clamped stiffeners/girders may be taken from Figure A.9-9

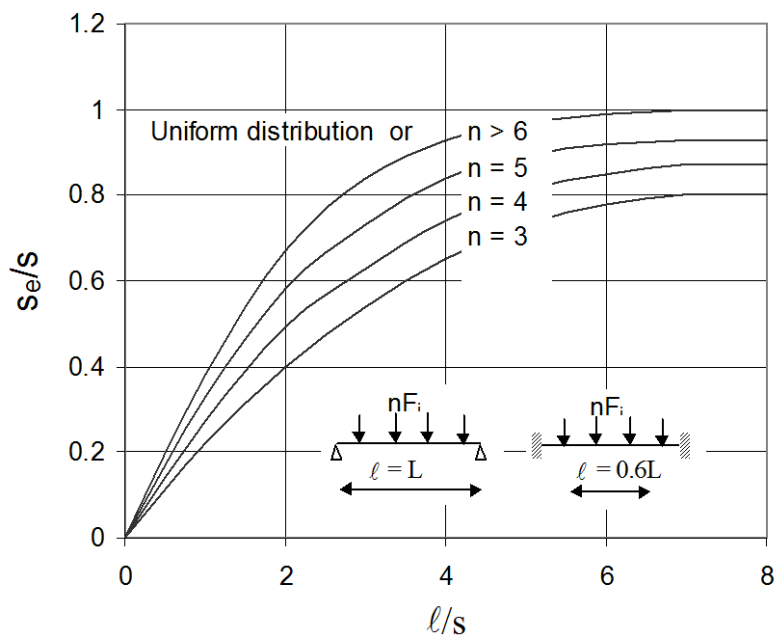


Figure A.9-9 Effective flange for stiffeners and girders in the elastic range

Comm. A.6.9.6 Ductility limits

The table is taken from Reference /4/. The values are based upon a limiting strain, elasto.plastic material and cross-sectional shape factor 1.12 for beams and 1.5 for plates. Strain hardening and any membrane effect will increase the effective ductility ratio. The values are likely to be conservative.

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DESIGN OF STEEL STRUCTURES
ANNEX K
SPECIAL DESIGN PROVISIONS FOR JACKETS

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K.1 GENERAL

K.1.1 Introduction

Annex K This Annex to NORSOK N-004 is intended to give guidance on how jacket steel structures should be designed according to the provisions of relevant NORSOK standards. It is intended to give guidance to how the various standards should be applied for jacket structures, how important parameters should be selected and to give additional requirements especially relevant for jacket structures.

K.1.2 Definitions

Jacket:	A welded tubular space frame structure consisting of vertical or battered legs supported by a lateral bracing system. The jacket is designed to support the topside facilities, provide supports for conductors, risers, and other appurtenances and serve as a template for the foundation system.
Bottle leg:	Leg section with larger diameter to thickness ratio used as buoyancy compartment during installation and to facilitate effective pile cluster design.
Foundation pile:	Steel tubular driven into the soil and fixed to the jacket structure for transference of global actions.
Pile cluster:	Pile sleeves for foundation piles arranged in groups with shear connections to jacket legs. Basis for mudmats and skirts.
Bucket foundation:	Steel plate construction integrated to bottom of jacket leg penetrating into soil for fixing platform to ground.
Multilegged jacket:	A jacket with more than 4 legs.

K.1.3 Design for non-operational phases

The jacket shall be designed to resist actions associated with conditions that will occur during stages from fabrication yard to the stage where the platform is ready for operation at final location. Such stages are transportation at the fabrication yard and load-out, sea transportation, installation, mating and hook-up. Abandonment of the jacket shall be planned for in design.

K.1.4 Design for operational phases

The jacket shall be designed to resist actions caused by gravity, wind, waves and current, earthquake and accidental actions that may occur during its service life.

Each mode of operation of the platform, such as drilling, production, work over or combinations thereof should be considered.

The fatigue requirements as given in Section 8 shall be fulfilled.

K.2 STRUCTURAL CLASSIFICATION

K.2.1 Structural classification

Selection of steel quality level and requirements for inspection of welds shall be based on a systematic classification of welded joints according to the structural significance and complexity of joints.

The main criterion for decision of design class (DC) of welded joints is the significance with respect to joint complexity and the consequences of failure of the joint. In addition the stress predictability (complexity) will influence the DC selection. Classification of DC for a typical 4-legged jacket is given in Table K.2-1 and Figure K.2-1.

A practical limit for water depths for which in-service inspection can be performed by use of diver technology may be set to 150 m. Consequently, an upgrade of one DC class should be specified for connections below 150 m water depths.

Table K.2-1 Typical design classes in jackets

Joint/ Component	Design Class DC	Steel Quality Level S.Q.L	Inspection Category Note a)	Comments
Legs and main bracing system				
Leg nodes & cones / Butt welds	2	(I) or II	A or B B	Note c), Note d)
Legs nodes & cones / Longitud. welds				
Leg strakes / Butt welds	2	II	A or B B	Note c), Note d)
Leg strakes / Longitudinal welds				
Lift-nodes / complex	1	I	A or B	
Lift-nodes / simple	2	(I) or II	A or B	
Nodes in vertical bracing / Butt welds	4	II	B C	
Nodes in vertical bracing / Longitudinal welds				
Vertical bracing / Butt welds	4	II	B C	
Vertical bracing / Longitud. welds				
Bottle leg / Butt welds	2	II	A or B B	
Bottle leg / Longitudinal welds				
Horizontal bracing / All welds	4	III	B or C B or C	
Nodes horizontal bracings / All welds				
Watertight diaphragms / All welds	2	II	A or B	
Ring stiffeners, main nodes / All welds		(I) or II	A	
Ring stiffeners bottle leg / All welds		(I) or II	A	
Other stiffening / All welds	5	III	D or E	
Foundation system				
Mudmat & yoke plate incl. stiffening / All welds	4	III	C	Note e)
Skirts & bucket foundation plates incl. stiffening / All welds	4	III	C	
Shear plates / All welds	4	III	C	Note e)
Pile sleeves / All welds	4	III	C	Note e)
Pile sleeve catcher, cone & spacers	5	IV	D or E	
Piles, top part / Butt welds	4	III	A	Note e)
Piles, top part / Longitud. welds				
Piles, remaining / All welds				
Appurtenances and outfitting steel				
Riser guides / All welds	4	III	B or C	Note b)
J-tubes & supports / All welds	4	III		
Conductor support / All welds	4	III		
Caissons & support / All welds	5	IV		
Outfitting / Butt welds	4 or 5	III or IV	D or E	Note b)
Outfitting / Part-Pen. & Fillets				

Notes:

- Local areas of welds with high utilisation shall be marked with frames showing areas for mandatory NDT when partial NDT are selected. Inspection categories depending on access for in-service inspection and repair.
- Outfitting structures are normally of minor importance for the structural safety and integrity. However, in certain cases the operational safety is directly influenced by the outfitting and special assessment is required in design and fabrication.
A typical example is guides and supports for gas risers.
- If multilegged jacket with corner legs supporting a foundation systems;
Upper part of corner legs and inner legs: DC 4, III, B or C
Lower part of corner legs: DC 2, II, A or B
- If multilegged jacket where each leg supporting a foundation system: DC 4, III, B or C
- If one or two pile(s) per leg: DC2, II.

When the design class is defined the material shall be selected according to Section 5.
The drawings shall indicate the inspection category for all welds according to Section 5.

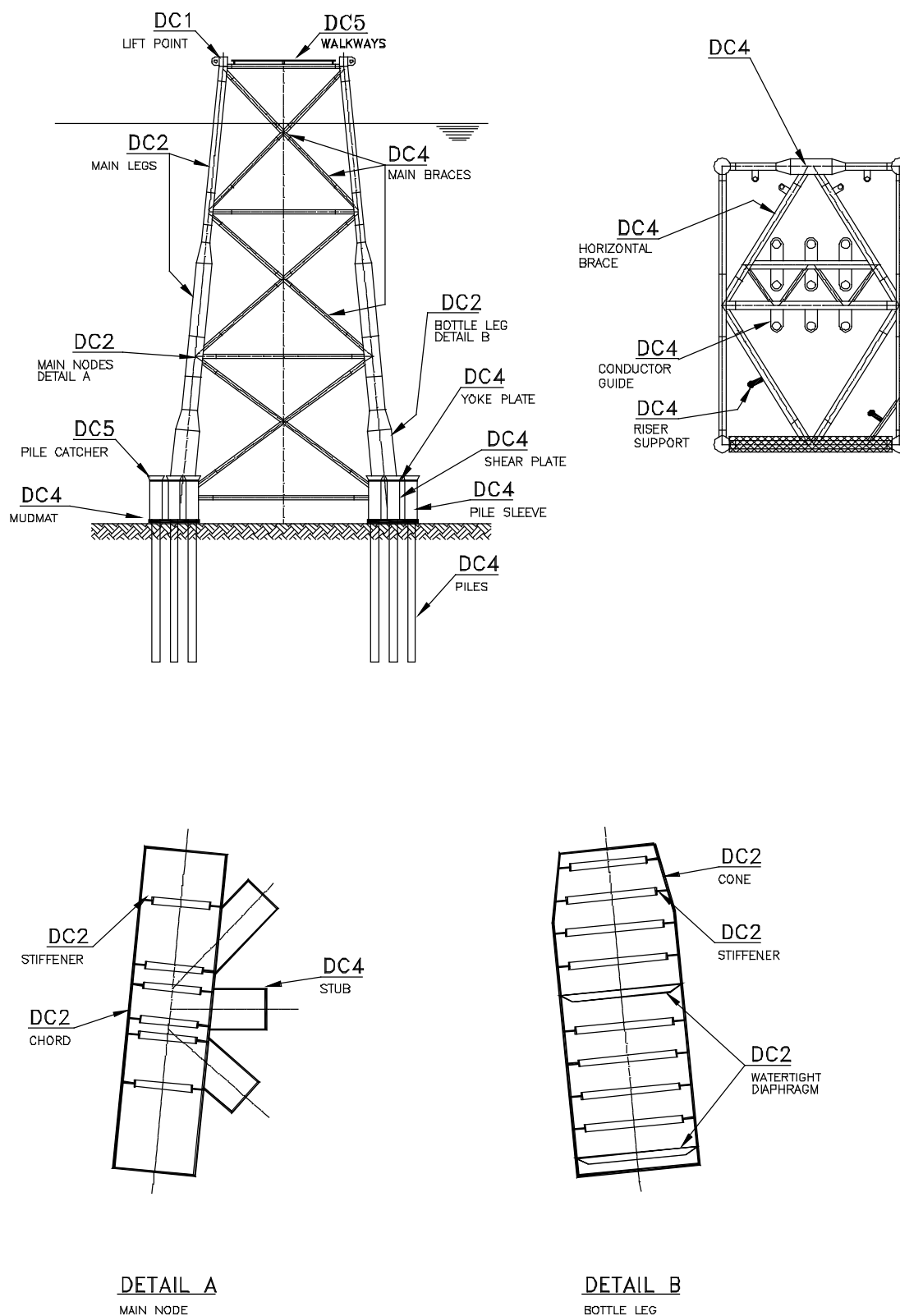


Figure K.2-1 Typical design classes. 4-legged jacket with pileclusters.

Table K.2-2 summarises the minimum design class and steel quality level for different parts of a typical seabed support and protection structure of moderate size. Severe criticality and severe fatigue utilisation may lead to more stringent assessment and requirements.

Table K.2-2 Typical design classes in subsea structures

<i>Joint/ Component</i>	<i>Design Class</i>	<i>Steel Quality-level S.Q.L Note a)</i>	<i>Inspection Category Note b)</i>	<i>Comments</i>
Main structure				
Corner legs – Bumper frame	DC 4	III	C or B	
Bottom frame	DC 4	III	C or B	
Padeye and padeye welds	DC 4 or 2	III or II	A or B	
Bracing and brace welds	DC 5	IV	C or B	
Pipes	DC 5	IV	C or B	
Hinge plate, butt welds	DC 5	IV	B	
Stiffeners, part pen & fillets			C	
Foundation				
Mudmat & skirts, butt welds	DC 4	III	B	
Part pen & fillets			C	
Plate girders incl. stiffeners	DC 5	IV		
Butt welds			C	
Part pen & fillets			D	
Ventilation & reinforcements	DC 5	IV	D	
Hatch				
Pipes	DC 5	IV	D or C	
Beams, butt welds	DC 5	IV	C	
Stiffeners, part pen & fillets			D	
Hinges (plates & bolt)	DC 5	III	C or B	
All other steel				
Outfitting, butt welds	DC 5	IV	D or E	Note c
Outfitting, part. pen & fillets	DC 5	IV	E	Note c

Notes:

- In cases with through thickness stress occur Z-quality steel should be evaluated, as an alternative additional testing may be performed
- Local areas of welds with high utilisation shall be marked with frames showing areas for mandatory NDT when partial NDT are selected.
- Outfitting structures are normally of minor importance for the structural safety and integrity. However, in certain cases the operational safety is directly influenced by the outfitting and special assessment is required in design and fabrication.

The actual MDS selection should be in compliance with M-120.

K.3 DESIGN ACTIONS

K.3.1 General

Characteristic actions shall be used as reference actions in the partial coefficient method.

Design actions are in general defined in NORSOK N-003.

Applicable action categories and combinations of actions relevant for jacket design are given in this Section.

K.3.2 Permanent action

Permanent actions are actions that will not vary in magnitude, position or direction during the period considered. Permanent actions relevant for jacket design are:

- mass of jacket incl. piles above mudline
- mass of appurtenances supported by the jacket and/or topside
- mass of permanently installed topside such as accommodation, drilling and production equipment
- buoyancy and hydrostatic pressure from sea water
- mass of marine growth

For design of a jacket, the characteristic value of permanent actions representing the mass of steel to be designed shall include contingency factors. The contingency plan shall reflect the uncertainties in the weight and material estimates as expressed by the stage in the calculations process.

At the detail design stage, the characteristic values of permanent actions are to be verified by accurate measurements (weight control) or calculated based upon accurate data.

K.3.3 Variable action

Variable actions on deck areas are actions that may vary in magnitude, position and direction during the period under consideration.

Normally, a jacket design is based upon budget weight of topside and a specified envelope of centre of gravity to account for variable actions on deck areas.

K.3.4 Deformation action

Deformation actions are actions due to deformations applied to the structure.

Deformation actions relevant for jacket design are:

- pre-stressing
- temperature
- barge deflections
- differential settlements
- uneven seabed
- field subsidence effects

K.3.5 Environmental action

K.3.5.1 General

The characteristic value of an environmental action is the maximum or minimum value (whichever is the most unfavourable) corresponding to an action effect with a prescribed probability of exceedance.

The environmental conditions should be described using relevant data for the relevant period and areas in which the jacket is to be fabricated, transported, installed and operated.

The long-term variation of environmental phenomena such as wind, waves and current should be described by recognised statistical distributions relevant to the environmental parameter considered. Information on the joint probability of the various environmental actions may be taken into account if such information is available and can be adequately documented.

Details of environmental actions are found in NORSOK N-003.

K.3.5.2 Wave and current action

Wave actions may be described either by deterministic or statistical methods. Normally, a deterministic analysis method is used for jacket design.

In deterministic design analysis based on regular wave formulation, the wave is described by wave period, wave height and water depth. Stokes 5th order wave kinematic theory shall be used.

In stochastic wave description, the short-term irregular sea states are described by means of wave energy spectra which are normally characterised by significant wave height (H_s), and average zero-up-crossing period (T_z), or spectral peak period (T_p).

Wave and current actions on the jacket structure are normally calculated by use of Morison's equation. Member drag (C_D) and inertia (C_M) coefficients shall be established according to NORSOK N-003.

Shielding effects may be taken into account where the presence of such effects can be adequately documented, (NORSOK N-003).

Where elements are closely grouped (e.g. conductors), solidification effects shall be considered.

The design current velocity and profile is to be selected using site specific statistics. The current applied in design shall include both tidal and wind induced effects. The current profile is normally defined up to still water level. In combination with waves, the profile shall be adjusted such that the flux of the current is kept constant.

K.3.5.3 Wind action

Extreme values of wind speeds are to be expressed in terms of most probable largest values with their corresponding recurrence periods.

Design of jacket and foundation shall be based upon the 1 hour extreme mean wind defined at a reference height of 10m above still water level. The 1 minute wind gust parameter shall be used when wind action is applied in combination with the wave, and the wave action being the dominating action.

Relevant scaling methods to obtain averaged wind speeds and wind height profiles are given in NORSOK N-003.

K.3.5.4 Earthquake action

For earthquake analyses, ground motions may be defined either as response spectra or time histories.

The selection of method depends on the actual problem being considered.

For deep-water jacket structures with long fundamental periods and significant soil-structure interaction effects, time-history analysis is normally recommended.

If inelastic material behaviour is considered, time-history methods are normally necessary.

When performing time-history earthquake analysis, the response of the structure/foundation system shall be analysed for a representative set of time histories.

In areas with small or moderate earthquake activity, the ULS check may be omitted and the earthquake analysis may be limited to the ALS check only.

K.3.5.5 Ice

If the structure is to be located in an area where snow or icing may accumulate, icebergs, or sea-ice may develop or drift, actions from such phenomena shall be taken into account where relevant, ref. NORSOK N-003.

K.3.6 Accidental action

Accidental actions are actions related to abnormal operation or technical failure.

Details of accidental actions are found in NORSOK N-003.

Accidental actions relevant for jacket design are:

- impact from vessel
- dropped object from crane handling
- pool fire at sea
- extreme environmental actions

The determination of accidental actions is normally to be based on a risk analysis.

K.3.7 Fatigue actions

Repetitive actions, which may lead to possible significant fatigue damage, shall be evaluated. The following listed sources of fatigue actions shall, where relevant, be considered:

- waves (including those actions caused by slamming and variable (dynamic) pressures).
- wind (especially when vortex induced vibrations may occur), e.g. during fabrication, installation and operation.
- currents (especially when vortex induced vibrations may occur).

The effects of both local and global dynamic response shall be properly accounted for when determining response distributions related to fatigue actions.

K.3.8 Combination of actions

Action coefficients and combinations of actions for the different limit states are in general given in NORSOK N-003.

The Serviceability Limit State (SLS) need not normally to be considered in jacket design if not special functional requirements are specified. Such functional requirements could be distance to other facilities (bridge connections, drilling rig) etc.

K.3.9 Vortex shedding

Vortex-induced action and vibrations with regard to wind, current and waves shall be investigated and taken into account where relevant.

Any contribution to fatigue damage from vortex shedding shall be added to damage from drag and inertia actions from the wave, (NORSOK N-003). Relevant hot-spot locations may be considered.

K.3.10 Wave slamming

Horizontal or near-horizontal members that may be subjected to wave slamming shall be designed to resist impact forces caused by such effects.

Any contribution to fatigue damage from wave slamming shall be added to damage from drag and inertia actions from the wave, (NORSOK N-003). Relevant hot-spot locations may be considered.

K.4 GLOBAL RESPONSE ANALYSES

K.4.1 General

The selected method of response analysis is dependent on the design condition; ULS, FLS or ALS. Due attention shall be paid to the dynamic behaviour of the structure including possible non-linearities in action and response.

K.4.2 Dynamic effects

Dynamic effects are present in all jacket structures. However, the significance of the dynamic effects is dependent upon stiffness, soil-structure interaction, mass and mass distribution and the structural configuration. E.g. a complex or wide configured space frame structure will experience different inertia actions than a narrow or column type structure due to influence on the natural period.

For traditional jacket platforms with natural periods less than 3 seconds, the inertial actions may be neglected and a quasi-static analysis will suffice provided that the provisions in K4.4 are adhered to.

The inertial action is established by a global dynamic analysis. The magnitude of the inertial action can be a direct result of the global dynamic analysis, or can be derived from the quasi-static analysis and the Dynamic Amplification Factor (DAF) from the global dynamic analysis.

Wave and other time varying actions should be given realistic representations of the frequency content of the action. Time history methods using random waves are preferred. Frequency domain methods may be used for the global dynamic analysis both for ULS and FLS, provided the linearization of the drag force can be justified.

The random waves should originate from one or more wave spectra that are plausible conditions for producing the design wave defined shape.

K.4.3 Analysis modelling

Internal forces in members should be determined by three-dimensional structural analysis.

Analytical models utilised in the global analyses of a platform should adequately describe the properties of the actual structure including the foundation.

The non-linear behaviour of axial and lateral soil-foundation system should be modelled explicitly to ensure action-deflection compatibility between the structure and the soil-foundation system.

The structural members of the framework may be modelled using one or more beam elements for each span between nodes. The number of beam elements depends on the element formulation, distribution of actions and potential for local dynamic response. Major eccentricities of action carrying members should be assessed and incorporated in the model. Face-to-face length may be modelled by stiff-end formulation of the chord.

For analyses of the operational design phases (static strength), any corrosion allowance on members should be subtracted in the stiffness and stress calculations, but included in the wave action calculations. For fatigue analysis, half the corrosion allowance thickness should be included both for action, stiffness and stress calculations.

Determination of fatigue lives in pile cluster connections should be determined based on stresses derived from a local finite element analysis.

Appurtenances such as conductors, J-tubes and risers, caisson, ladders and stairs, launch box, boat landing, guides and anodes should be considered for inclusion in the hydrodynamic model of the structure. Depending upon the type and number, appurtenances can significantly increase the global wave forces. In addition, forces on some appurtenances may be important for local member design. Appurtenances not welded to main structure are generally modelled by non-structural members that only contribute as equivalent wave forces.

The increase in hydrodynamic- and gravity actions caused by marine growth shall be accounted for.

K.4.4 Design conditions

K.4.4.1 General

The platform shall be designed to resist gravity actions, wave, current and wind actions, earthquake and accidental actions that may occur during its service life.

K.4.4.2 In-place ULS analysis

Requirements concerning in-place ULS capacity checks are given in Section 6.

Each mode of operation of the platform, such as drilling, production, work-over or combination thereof shall be considered. The stiffness of the deck structure shall be modelled in sufficient detail to ensure compatibility between the deck design and the jacket design.

Studies should be performed to establish maximum base shear of wave and current actions for dimensioning of jacket bracings. Maximum overturning moments shall be established for dimensioning of jacket legs and foundation system. Diagonal approach directions normal to an axis through adjacent legs should also be considered in search for maximum leg and foundation reactions. The studies shall be undertaken by varying wave approach directions, wave periods and water depths. Detail design analysis should be based on minimum eight wave approach directions. A reduced number of approach directions may be considered for early stage design analysis for jackets which are symmetric about the two vertical axes.

Maximum local member actions may occur for wave positions other than that causing the maximum global force. Horizontal members close to still water level shall be checked both for maximum vertical and horizontal wave particle velocities. The effect of varying buoyancy shall also be included.

The choice of an appropriate design wave theory is to be based on particular considerations for the problem in question. Stokes' 5th order wave theory is normally used for jacket design. However, for low water depth jacket locations, shallow water effects shall be assessed.

DAF's should be established by use of time history analyses for structures where the dynamic effects are important.

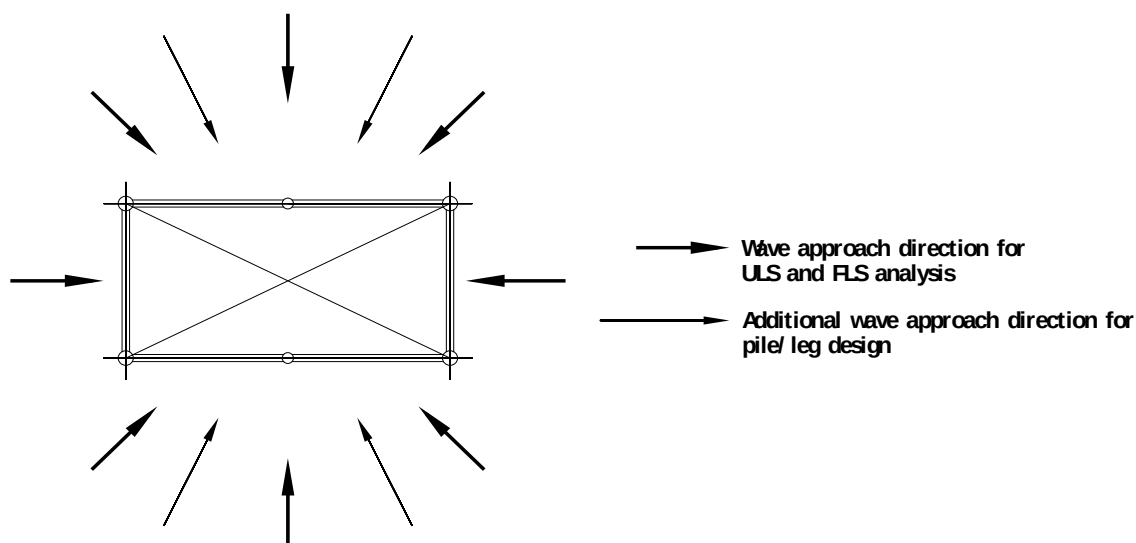


Figure K.4-1 Recommended wave approach directions for ULS and FLS analysis

K.4.4.3 Fatigue analysis

General requirements concerning fatigue analysis of steel structures are given in Section 8. Special considerations and details of fatigue strength analysis are given in Annex C.

Fatigue analyses should include all relevant actions contributing to the fatigue damage both in non-operational and operational design conditions. Local action effects due to wave slamming, vortex shedding are to be included when calculating fatigue damage if relevant.

Whilst jackets in low to moderate water depths are not normally sensitive to dynamic effects, non-linearities associated with wave theory and free-surface effects may be important. A deterministic analysis is normally recommended for such jackets. In the detail design phase, the deterministic analysis should include eight wave approach directions and at least four wave heights from each direction. Wave forces should be

calculated for at least ten positions in each wave. In performing a deterministic wave analysis, due attention should be paid to the choice of wave periods if such data are not explicitly specified, or can be determined based upon wave scatter diagrams relevant for the location.

In lack of site specific data, the wave periods shall be determined based on a wave steepness of 1/20 (NORSOK N-003).

For deep water jackets and jackets where the dynamic effects are important, a fatigue analysis in the frequency domain (dynamic stochastic analysis) is recommended.

Frequency domain action calculations are carried out in order to determine hydrodynamic transfer functions for member force intensities. The transfer functions are expressed as complex numbers in order to describe the phase lags between the action variable and the incoming wave

Studies are to be performed to investigate wave actions for a range of periods in order to ensure a sufficient accuracy of the response. For a stochastic fatigue analysis, it is important to select periods such that response amplifications and cancellations are included. Also selection of wave periods in relation to the platform fundamental period of vibration is important. The number of periods included in the analysis should not be less than 30, and be in the range from $T = 2$ sec. to at least $T = 20$ sec.

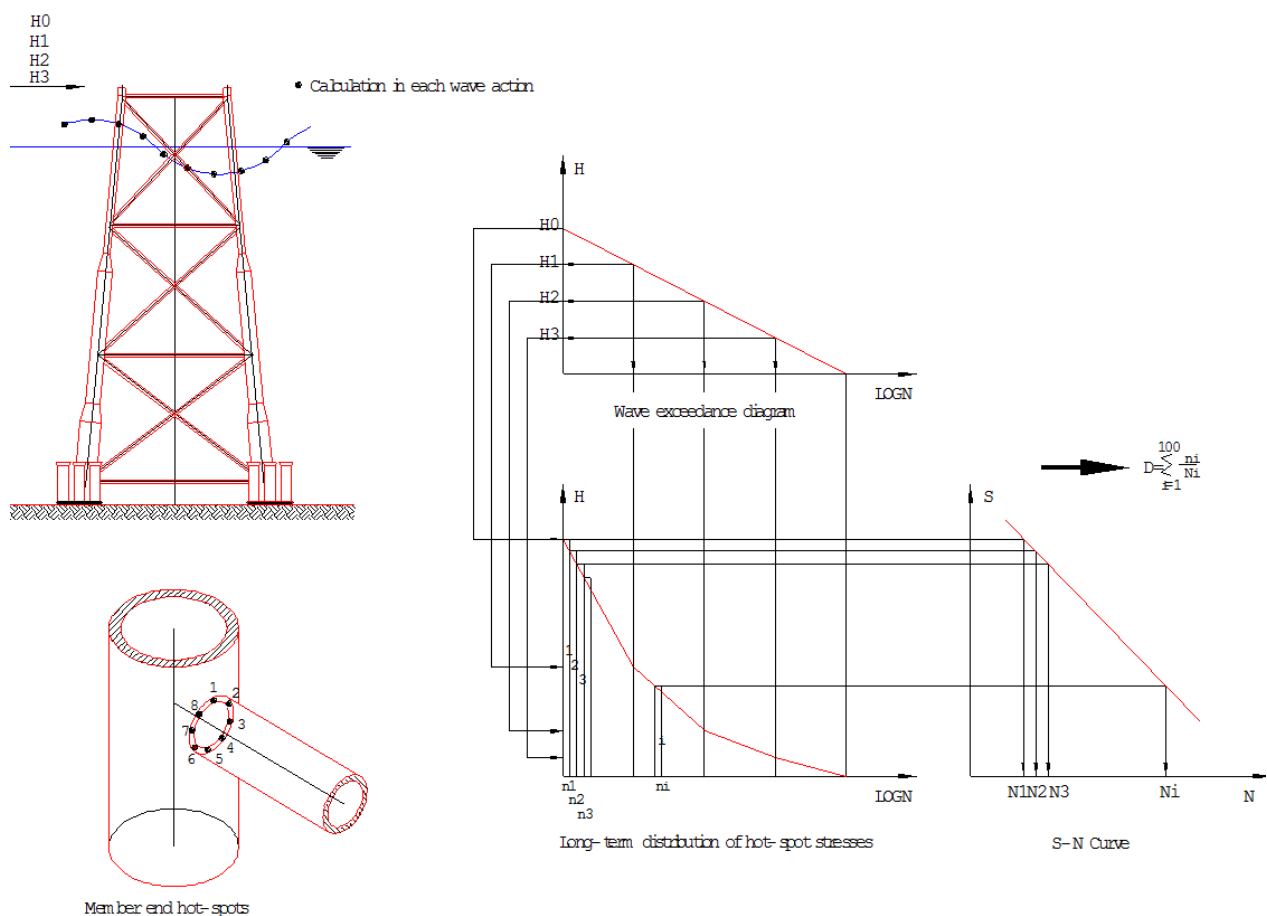


Figure K.4-2 Deterministic fatigue analysis procedure

Short crested waves (angular distribution of wave energy) may be taken into account if such effects are present at the location. When applying short crested waves, an increased number of wave approach directions should be used, normally not less than 12.

The dynamic analysis can be carried out by a modal superposition analysis, direct frequency response analysis or by mode syntethesis techniques.

Traditional jackets are drag dominated structures. For drag dominated structures stochastic analysis based upon linear extrapolation techniques may significantly underestimate action effects. When the effect of drag forces on the expected fatigue damage and the expected extreme responses are to be assessed, Morison-type wave actions are to be based on relevant non-linear models.

If practical, horizontal plan elevations (or complex systems) should be designed to be located away from the still water level or splash zone.

Structure-to-ground connections in the analysis shall be selected to adequately represent the response of the foundations. Structure-to-ground connections may normally be simulated by linear stiffness matrices. To linearize the actually non-linear soil response, these matrices should be developed based on a wave height which contributes significantly to the fatigue damage. The matrices shall account for the correlation between the rotational and translation degrees of freedom, which may be important for proper calculation of fatigue lives in the lower part of the structure.

Structural components in the jacket shall be classified according to consequences of failure and accessibility for inspection and repair as outlined in Section 8.

Structures or structural parts located in water depths below 150 m shall be considered as being not accessible for inspection and repair.

Fatigue design factors for typical components in jackets are given in Table K.4-1 and Figure K.4-3.

Table K.4-1 Fatigue design factors in jackets

Classification of structural components based on damage consequence	Not accessible for inspection and repair or in the splash zone	Accessible for inspection, maintenance and repair, and where inspections or maintenance is planned	
		Below splash zone	Above splash zone
• Brace/stub to chord welds in main loadtransferring joints in vertical plans, • Chord/cone to leg welds, between leg sections, • Brace to stub and brace to brace welds in main loadtransferring members in vertical plans, • Shear plates and yoke plates incl. stiffening • Piles and bucket foundation plates incl. stiffening	10	3	2
• Brace/stub to chord welds in joints in horisontal plans, • Chord/cone to brace welds and welds between sections in horisontal plans, • Appurtenance supports, • Anodes, doubler plates, • Outfitting steel	3	2	1

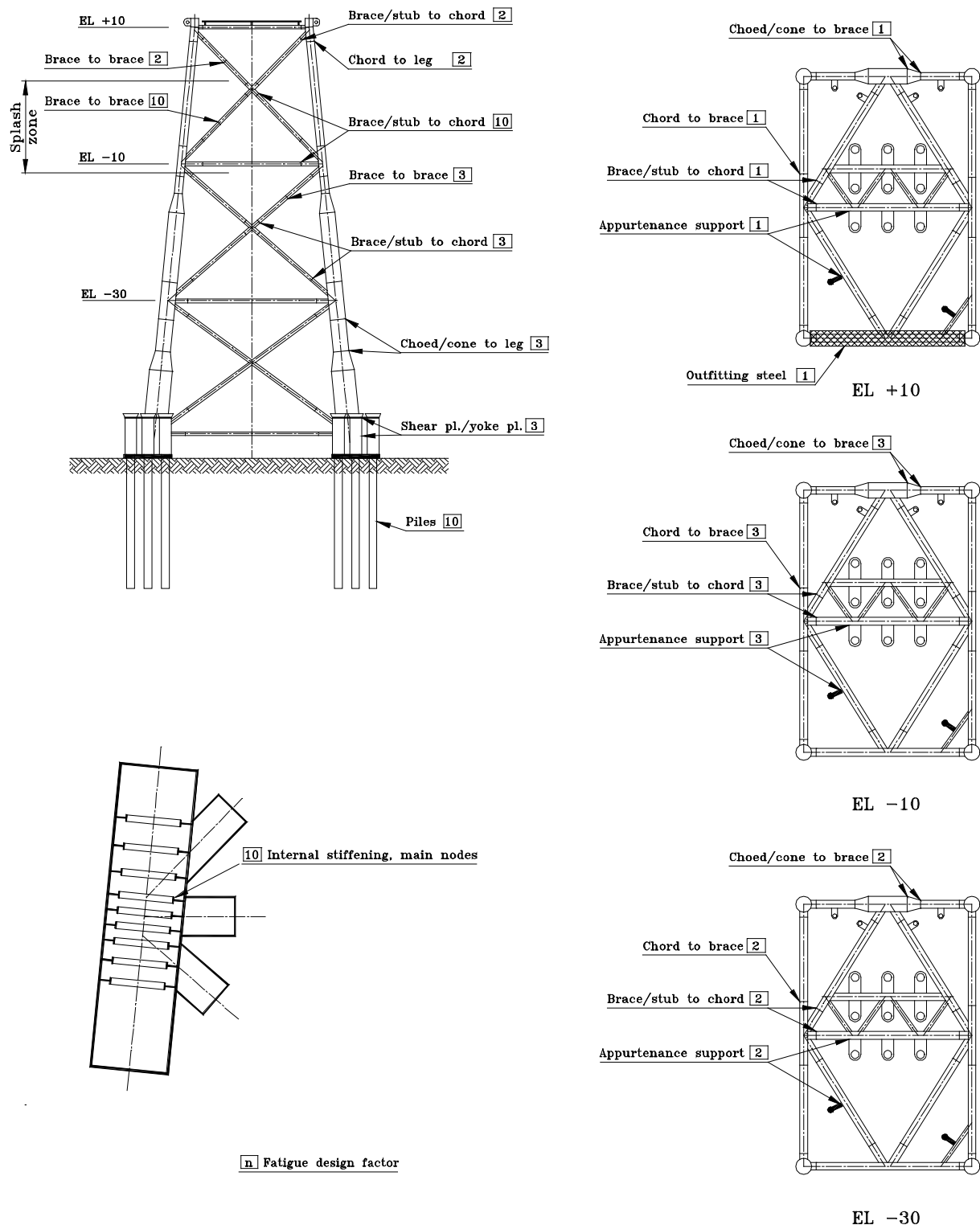


Figure K.4-3 Typical fatigue design factors in jackets

K.4.4.4 Accidental analysis

General requirements concerning the Accidental Limit State (ALS) of steel structures are given in Section 9. The jacket is to be designed to be damage tolerant, i.e. credible accidental damages or events should not cause loss of global structural integrity.

A credible collision or dropped object against a bracing member or a nodal brace joint will reduce the action carrying ability of the member or the joint. Such members or joints are thus to be assumed non-effective when the global strength of the platform (residual strength) is to be assessed for combinations of design actions in the ALS condition. A credible collision or dropped object against a leg member will normally cause limited damages, and the residual strength of the platform is to be assessed taking due account of the residual strength of the damaged member for combination of design actions in the ALS condition. Details may be found in Annex A.

Pool fire analyses are normally performed to establish requirements to the platform passive and active fire protection system.

For consideration of the 10^{-4} wave action and requirement to air gap see Norsok N-003.

K.4.4.5 Earthquake analysis

General requirements concerning earthquake analyses are given in NORSOK N-003.

Deck facilities that may respond dynamically to earthquake excitation (flare boom, derrick) are to be included in the structural model where relevant, in order to account for interaction effects due to correlation between closely spaced modes of vibration.

Earthquake analysis may be performed using response spectrum analysis or time history analysis. When the response spectrum method is used, the design spectrum shall be applied in one of the orthogonal directions parallel to a main structural axis. 2/3 of the design spectrum shall be applied in the other orthogonal direction and 2/3 in the vertical direction. The complete quadratic combination (CQC) method should be used for combining modal responses. The square root of the sum of the squares (SRSS) method should be used for combining directional responses.

The model used for earthquake analysis should satisfactorily simulate the behaviour of the actual structure. The number of vibration modes in the analysis should represent at least 90% of the total response energy of all modes. The number of modes to be included in the analysis is normally to be determined by a parametric study. Normally, 15-20 modes are sufficient to ensure an adequate representation of all response quantities. The number of modes may be larger if topside structures such as flare booms and derricks are incorporated in the global model and if their dynamic behaviour is required

Topside structures may normally be included in a simplified manner provided that such modelling will properly simulate the global stiffness and mass distributions.

Structure-to-ground connections are normally simulated by linear stiffness matrices. For jackets located in shallow to moderate water depths, conservative response values are usually achieved by a stiff soil assumption. However, for deep water jackets this may not be conservative.

Damping properties should be adequately assessed and included in the analysis as appropriate. In the absence of more accurate information, a modal damping ratio of 5 % of critical may be used. This value covers material damping and hydrodynamic damping, as well as radiation and hysteretic soil damping. Other damping ratios may be used where substantiated data exists.

K.4.4.6 Installation analysis

Transport and installation design and operation shall comply with the requirements given in the VMO standard.

The recurrence periods to be considered for determination of environmental actions shall comply with the requirements given in NORSOK N-001.

K.5 SPECIAL DESIGNS

K.5.1 Member design

Reference is made to Section 6 for static strength requirements to tubular members.

Buckling factors for X-bracing's may be determined according to procedures which take into account the degree of lateral support furnished to the primary compression member by the cross member (tension member) and member end restraints.

Thickness transitions should be made by flushing inside of tubular to achieve good fatigue capabilities. Reference is made to DNV-RP-C203 for fatigue requirements to butt welds.

K.5.2 Tubular connections

Reference is made to Section 6 and 8 for static strength – and fatigue requirements to tubular member connections and conical transitions.

K.5.3 Grouted connection

K.5.3.1 General

Grouted pile connections shall be designed to satisfactorily transfer the design loads from the pile sleeve to the pile as shown in Figure K.5-1. The grout packer may be placed above or below the lower yoke plate as indicated in Figure K.5-2. The connection may be analysed by using a load model as shown in Figure K.5-3. The following failure modes of grouted pile to sleeve connections need to be considered:

- Failure of grout to pile interface shear due to axial load and torsional moment (ULS and ALS).
- Failure of the grout due to compressive stresses at the lower end of the grout due to bending moment and shear force in the pile (ULS and ALS).
- Fatigue of the grouted connection for alternating interface shear stress due to axial load and bending moment in the pile (FLS).
- Fatigue of the grout due to contact pressure and friction forces creating shear stresses at the lower end caused by bending moment and shear force in the pile (FLS).

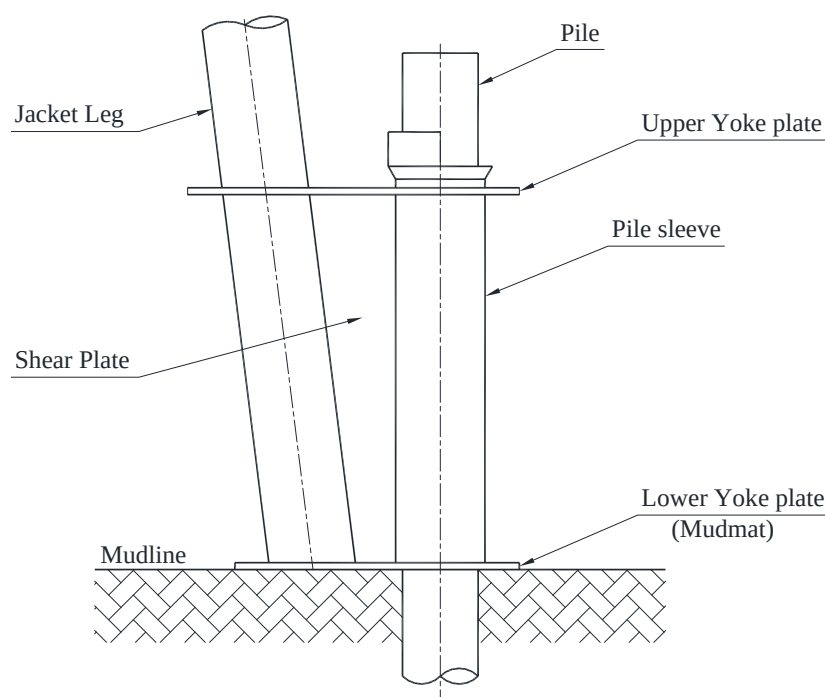


Figure K.5-1 Terms for typical pile-sleeve connections

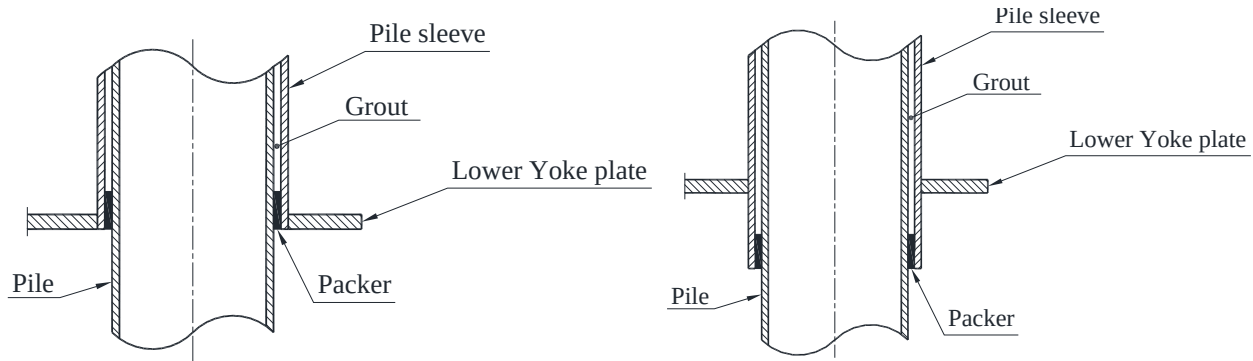


Figure K.5-2 The left figure shows grout termination above Lower Yoke plate and the right figure shows grout termination below Lower Yoke plate

The recommendations for check of the above failure modes for pile sleeve connections with circular hoop or helix curved strings of weld beads or bars denoted shear keys are described in Section K.5.3.2 to K.5.3.5

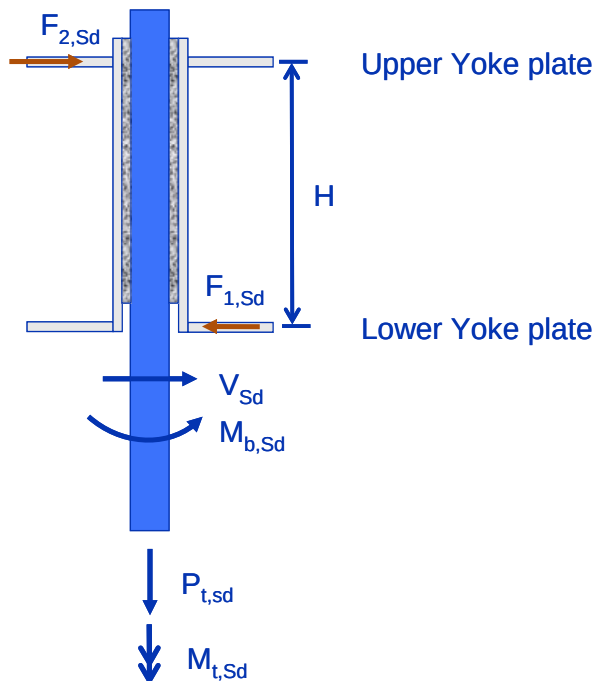


Figure K.5-3 Model for calculation of forces in grouted pile-sleeve connections

K.5.3.2 Failure of the grout to pile connection due to interface shear from axial load and torsional moment (ULS and ALS)

When a grouted connection is subjected to combined axial force and torsional moment, the interface shear stress shall be taken as the result of the component stresses caused by axial force and torsional moment calculated at the outer surface of the inner member.

The design interface stress due to axial force, $\tau_{ba,Sd}$, is defined by:

$$\tau_{ba,Sd} = \frac{N_{Sd}}{\pi \cdot D_p \cdot L_e} \quad (\text{K.5.1})$$

where

N_{Sd}	=	design axial force [N]
D_p	=	outside diameter of pile [mm]
L_e	=	effective grouted connection length [mm]

In calculating the effective grouted connection length, L_e , the following non-structural lengths shall be subtracted from the connection's nominal gross grouted length:

1. Where setting of a grout plug is the primary means of sealing, or is the contingency sealing method in the event of packer failure, the grout plug length shall be considered as non-structural.
2. To allow for potential weak interface zones, grout slump, etc. at each end of the connection, the greater of the following grouted lengths shall be considered as non-structural:
 - two thickness of the grout annulus, $2t_g$
 - one shear key spacing, s , if shear keys are used.
3. Any grouted length that is not certain to contribute effectively to the connection capacity, shall be considered as non-structural (e.g. when shear keys are used, the implications of possible over and under driving of piles shall be considered in relation to the number of shear keys present in the grouted length).
4. For pile-sleeve connections carrying significant bending moments a distance of $\sqrt{D_p \cdot t_p}$ above and below the lower yoke plate should be without shear keys.

The design interface transfer stress due to torsional moment, $\tau_{bt,Sd}$, is defined by:

$$\tau_{bt,Sd} = \frac{2M_{t,Sd}}{\pi \cdot D_p^2 \cdot L_e} \quad (K.5.2)$$

where

$M_{t,Sd}$ = design torsional moment on the connection.

The combined axial and torsional design interface shear is calculated as:

$$\tau_{b,Sd} = \sqrt{\tau_{ba,Sd}^2 + \tau_{bt,Sd}^2} \quad (K.5.3)$$

The characteristic interface strength for grout to steel interface with shear keys, is given by:

$$f_{bks} = \left(\frac{800}{D_p} + 140 \cdot \left(\frac{h}{s} \right)^{0.8} \right) \cdot C_s^{0.6} \cdot f_{ck}^{0.3} \quad (K.5.4)$$

The characteristic interface transfer strength for grout steel interface in the connection without shear keys, is given by:

$$f_{bkf} = \left(\frac{800}{D_p} \right) \cdot C_s^{0.6} \cdot f_{ck}^{0.3} \quad (K.5.5)$$

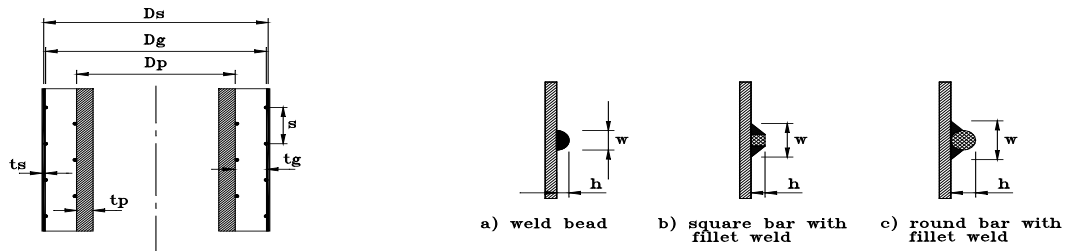
The characteristic interface transfer strength for grout matrix shear failure is given by:

$$f_{bkg} = \left(0.75 - 1.4 \cdot \left(\frac{h}{s} \right) \right) \cdot f_{ck}^{0.5} \quad (K.5.6)$$

where

s	=	shear key spacing (mm)
h	=	shear key height (mm)
C _s	=	radial stiffness factor
	=	$[(D_p / t_p) + (D_s / t_s)]^{-1} + (1 / m) \cdot (D_g / t_g)^{-1}$
f _{ck}	=	characteristic cube strength (MPa)
D _p	=	outside diameter of pile (mm)
t _p	=	wall thickness of pile (mm)
D _s	=	outside diameter of pile sleeve (mm)
t _s	=	wall thickness of pile sleeve (mm)
D _g	=	outside diameter of grout annulus (mm)
t _g	=	thickness of grout annulus (mm)
E	=	Young's modulus of elasticity for steel (MPa)
m	=	steel-grout elastic modular ratio (to be taken as 18 in lieu of actual data)

The inherent variability in the test data should be considered when calculating the characteristic strength if the capacity is based on test results.



GROUPED CONNECTION WITH SHEAR KEY

RECOMMENDED SHEAR KEY DETAILS

Figure K.5-4 Grouted connection with recommended shear key details

Equation (K.5.4), (K.5.5) and (K.5.6) are valid for uncoated tubulars with normal fabrication tolerances, where mill scale has been fully removed. The recommendations are valid for the following range:

80 MPa	≥	f _{ck}	≥	20 MPa
0.10	≥	h/s	≥	0.0
40	≥	D _p /t _p	≥	20
140	≥	D _s /t _s	≥	30
45	≥	D _g /t _g	≥	10
0.012	≥	h/D _p		
16	≥	D _p /s		
10	≥	L _e /D _p	≥	1

Connection with circular hoop shear keys should be checked as follows:

$$\tau_{ba,Sd} \leq \frac{f_{bks}}{\gamma_M} \quad (K.5.7)$$

$$\tau_{bt,Sd} \leq \frac{f_{bkf}}{\gamma_M} \quad (K.5.8)$$

$$\tau_{b,Sd} \leq \frac{f_{bkg}}{\gamma_M} \quad (K.5.9)$$

where

γ_M = material factor for interface transfer strength equal to 2.0 for ULS and 1.5 for ALS

If the check of torsion stresses according to (K.5.8) is not fulfilled the ULS checks can be made assuming redistribution of the pile torsion moment. This can be done by releasing the corresponding degree of freedom for the pile in the model. In addition a FLS check according to K.5.3.6 needs to be satisfied.

A shear key shall be a continuous hoop or a continuous helix. Where hoop shear keys are used, they shall be uniformly spaced, oriented perpendicular to the axis of the tube, and be of the same form, height and spacing on both the inner and outer tubes.

Where helical shear keys are used, the following additional limitation shall be applied:

$$s \leq D_p/2.5$$

and the characteristic interface transfer strength given by equations (K.5.4) and (K.5.6) shall be reduced by a factor of 0.75.

The possible movements between the inner and the outer steel tubular member during the 24 hour period after grouting shall be determined for the maximum expected sea states during that time, assuming that the grouted connection does not contribute to the stiffness of the system. For foundation pile-to-sleeve connections, this analysis shall be an on-bottom analysis of the structure with ungrouted piles.

If the expected relative axial movement at the grout steel interface exceeds 0.035% D_p during this period, the movements should be limited by e.g. installation of pile grippers.

K.5.3.3 Check of compressive stresses at the lower end of the grout due to bending moment and shear in the pile (ULS and ALS)

The compressive capacity of the grout shall be checked for forces in the Ultimate Limit States (ULS) and the Accidental limit States (ALS).

The compressive contact stress between the steel and the grout will create tensile stresses in the grout due to the shear stresses resulting from the friction forces at the grout to steel surfaces. It is considered acceptable that the grout cracks for tensile stresses during these limit states; however, the grout needs to transfer the forces from the sleeve to the pile throughout the storm that includes the dimensioning environmental load.

The following procedure assumes that the specified mechanical properties of the grout are met at all positions within the area assumed for carrying loads. If there is doubt about the grout quality that can be achieved at these locations, the calculations should be adjusted accordingly or the connection should be fitted with reinforcement bars according to K.5.3.7 in order to improve the robustness and ductility of the connection.

The design contact pressure between steel and grout can be obtained from

$$\sigma_{p, Sd} = C_A \frac{F_{1, Sd}}{\sqrt{D_p^3 t_p}} \quad (K.5.10)$$

Where:

$$F_{1, Sd} = V_{Sd} + M_{b, Sd} / H \quad (\text{See Figure K.5-3})$$

$$C_A = 2 \text{ for grout ending above lower yoke plate and without reinforcement steel (See Figure K.5-2)}$$

$$= 1 \text{ for grout ending a minimum distance } = \sqrt{D_p \cdot t_p} \text{ below lower yoke plate and without reinforcement steel}$$

$$= 1 \text{ for grout ending above lower yoke plate and with longitudinal reinforcement steel}$$

$$= 0.5 \text{ for grout ending a minimum distance } = \sqrt{D_p \cdot t_p} \text{ below lower yoke plate and with reinforcement steel.}$$

V_{Sd} = Pile resultant shear force with ULS or ALS load factors as appropriate

$M_{b,Sd}$ = Pile resultant bending moment with ULS or ALS load factors appropriate

The largest principal design stress can be calculated as

$$\sigma_{I,Sd} = \frac{\sigma_{p,Sd}}{2} \left(1 + \sqrt{1 + 4\mu^2} \right) \quad (\text{K.5.11})$$

where a friction coefficient between pile and grout should be taken as: $\mu = 0.7$

It should be checked that:

$$\sigma_{I,Sd} \leq \frac{f_{cN}}{\gamma_M} \quad (\text{K.5.12})$$

where

$$f_{cN} = 0.85 \cdot f_{ck} \left(1 - \frac{f_{ck} \cdot 0.85}{600} \right) \quad (\text{K.5.13})$$

f_{ck} = characteristic cube strength of grout in MPa

$\gamma_M = 1.5$ for ULS and 1.25 for ALS

When steel reinforcements are prescribed, the reinforcements should meet requirement given in K.5.3.7.

K.5.3.4 Fatigue of the grouted connection for alternating interface shear stress due to axial load and bending moment in the pile (FLS)

General

The maximum axial loads in piles of jacket platforms will normally be in compression for the case where permanent and variable loads add to the loads from environmental actions. However, as the capacity of grouted connections exposed to cyclic loads where the axial load changes from compression to tension during the load cycle are known to be significantly less than the static capacity it is necessary to check the conditions where the axial load in the pile is in tension.

As research data on the long-term capacity of grouted connections is scarce especially on effects from moment, there are no established methods for capacity assessment for pile-sleeve connections exposed for cyclic loads. Until adequate test data is available the following simplified fatigue check is proposed:

Calculate the maximum pile axial tension load ($P_{t,Sd}$) for 100 year loading with load factor $\gamma_f = 1.0$ considering minimum permanent and variable loads.

Calculate the axial tension capacity as:

$$P_{f,Rd} = \frac{0.3 f_{bks} \cdot D_p \cdot L_e}{\gamma_M} C_{PMred} \quad (\text{K.5.14})$$

where:

f_{bks} = characteristic interface strength for grout to steel

D_p = outside diameter of pile

L_e = effective grouted connection length, see K.5.3.2

γ_M = material factor = 2.0 for pile clusters with one or two piles and 1.5 for pile clusters with 3 or more piles if the integrity of the connection can be confirmed by inspection after severe storms.

C_{PMred} = Reduction factor for alternating moment.

$$C_{PMred} = 1 - \frac{D_p \cdot M_{PEnv,Sd}}{H \cdot M_{ref}} \quad (\text{K.5.15})$$

where:

$M_{P\ Env,Sd}$ = bending moment from 100 year environmental loads with load factor $\gamma_f = 1.0$
 $M_{ref} = W \cdot 0.001 \cdot E$
 E = Youngs modulus of the steel in the pile
 W = Elastic section modulus of the pile

The stiffness representation of the soil should represent the best estimate for cyclic loading.

Check if

$$P_{t,Sd} \leq P_{f,Rd} \quad (K.5.16)$$

If equation (K.5.16) is not fulfilled, the capacity from friction created by the forces from the minimum corresponding bending moment and shear force in the pile according to the calculation model shown in Figure K.5-3 can be added to obtain larger capacity. The reaction forces can be calculated as:

$$F_{1,Cor} = V_{Cor} + M_{Cor} / H \quad (K.5.17)$$

$$F_{2,Cor} = M_{Cor} / H$$

$$F_{3,Cor} = 2M_{t,Cor} / D_p$$

V_{Cor} and M_{Cor} are the minimum shear and bending moment that correspond with the tensile axial force $P_{t,Sd}$ and $M_{t,Cor}$ is the corresponding maximum torsional moment to be taken by the pile.

Then the net friction force can be derived as

$$F_{\mu,Rd} = \left(\sqrt{(F_{1,Cor} + F_{2,Cor})^2 - F_{3,Cor}^2} \right) \mu \quad (K.5.18)$$

where:

μ = coefficient of friction between pile and grout for the contact area at the upper and lower yoke plates = 0.4 for this assessment

Check if

$$P_{t,Sd} \leq F_{\mu,Rd} + P_{f,Rd} \quad (K.5.19)$$

K.5.3.5 Fatigue of the grout due compression and shear stresses at the lower end of the grout due to bending moment and shear in the pile (FLS)

The stress variations in the grout at the lower end is caused by cyclic bending moment and shear force in the pile giving compression stress in the grout and shear stress due to the friction caused by the sliding between the pile and the grout. For a grouted connection without steel reinforcement, one should limit these friction stresses such that they will not exceed the tensile capacity more than once during the life of the platform. This is considered to be achieved if the following requirements are met based on calculation of 100 year return period environmental loads in combination with permanent loads.

For fatigue the action effects are derived with a load factor $\gamma_f = 1.0$.

The tensile stress in the grout can be calculated as

$$\sigma_{II,Sd} = \frac{\sigma_{p,Sd}}{2} \left(1 - \sqrt{1 + 4\mu^2} \right) \quad (K.5.20)$$

where a friction coefficient between pile and grout should be taken as $\mu = 0.7$.

The design criterion reads:

$$|\sigma_{II, Sd}| \leq \frac{f_{tk}}{\gamma_M} \quad (K.5.21)$$

where

f_{tk} = the characteristic tensile strength according to DNV-OS-C502 determined from splitting tensile testing of the actual grout.

$\gamma_M = 1.25$ = material factor used in fatigue assessment for the grout.

If equation (K.5.21) is not fulfilled, the capacity can be increased by introduction of longitudinal and circumferential reinforcements. The fatigue capacity of the reinforced grouted section is for ordinary jacket pile-sleeve connections judged to be acceptable if the requirements stated in K.5.3.7 are met.

K.5.3.6 Fatigue check due to torsion

In cases where the ULS check according to K.5.3.2 fails in the check for interface shear stresses due to torsional moments and redistribution of the moment is assumed, the following check need to be carried out. This check is valid for connections with hoop shear keys.

The return period for the load should be taken as 100 year and the load factor $\gamma_f = 1.0$.

$$\tau_{btEnv100, Sd} \leq f_{bkf} \quad (K.5.22)$$

where

$$\tau_{btEnv100, Sd} = \frac{2M_{tEnv100, Sd}}{\pi \cdot D_p^2 \cdot L_e} \quad (K.5.23)$$

$M_{tEnv100, Sd}$ = Design torsional moment from the 100 year return period environmental actions

If this check is not satisfied it is recommended to increase the capacity against torsional moments e.g. by introduction of shear keys transverse to the direction of the interfaces shear stresses.

K.5.3.7 Requirements to ribbed steel reinforcement

If the grout strength and durability is increased by introduction of ribbed steel reinforcement bars, the following requirements to the steel reinforcement are assumed for the formulas given in this Annex:

- The steel reinforcements should be arranged in the longitudinal direction of the sleeve starting from the lower end of the grout over a length of minimum $2.5 \sqrt{D_p \cdot t_p}$ in case for grout ending above lower yoke plate and $5.0 \sqrt{D_p \cdot t_p}$ for the case when the grout extend below the lower yoke plate.
- The longitudinal reinforcements should be placed as close as possible to the pile wall.
- Each reinforcement bar should be bent and welded to the sleeve at the top and bottom for the full yield capacity of the bar.
- Longitudinal bar spacing should be in the range of 0.5 to 2.0 times the grout thickness.
- Ring bars to be arranged to secure the position of the longitudinal bars throughout all phases and should as a minimum meet ordinary code requirements to reinforcements for shrinkage (e.g. DNV-OS-C502).
- The required area of the longitudinal reinforcement can be calculated as

$$A_s = 0.5 \frac{\sigma_{p, Sd}}{f_{sd}} \mu t_g b \quad (K.5.24)$$

Where

$\sigma_{p, Sd}$	to be calculated according to Equation (K.5.10) using C_A for the case with reinforcements.	
t_g	= thickness of grout	
b	= distance between longitudinal reinforcement bars	
f_{sd}	$= f_{sy} / \gamma_M$	= design strength of the reinforcement bars
f_{sy}	= characteristic yield stress of the reinforcement bars	
γ_M	= 1.0	= material factor for reinforcement bars
A_s	= area of reinforcement bar	
μ	= friction coefficient	= 0.6

K.5.3.8 Considerations on in-service inspection

Generally the experience with grouted pile-sleeve connections is good and regular in-service inspections have not been considered to be necessary. However, it should be kept in mind that the long term performance of grouted sleeve connections is uncertain. An effective in-service inspection of these connections is usually difficult to execute as there is limited access. Especially if the requirements of this standard are not met it should be considered to implement measures at the design and construction phases that enable easy inspections to check that the connection is functioning as intended.

Presently, there are few adequate inspection methods for grouted pile-sleeve connections available, but it is expected that more suited inspection methods will be developed in the time to come.

K.5.4 Cast items

Cast joints are defined as joints formed using a casting process. They can be of any geometry and of variable wall thickness.

The design of a cast joint is normally done by use of FE analyses. An acceptable design approach for strength is to limit stresses in the joint due to factored actions to below yielding of the material using appropriate yield criteria. Additionally, the FE analyses should provide SCF's for fatigue life calculations. Often, this design process is carried out in conjunction with the manufacturer of the cast joints.

Fatigue requirements to cast nodes are given in DNV-RP-C203.

K.6 FOUNDATION DESIGN

K.6.1 General

K.6.1.1 Design principles

The principles of limit state design and the partial coefficient format shall be used. A general description of the method and the definition of the limit state categories are given in the ISO Standard 19900 /1/.

Material factors are to be applied to the soil shear strength as follows:

- for effective stress analysis, the tangent to the characteristic friction angle is to be divided by the material factor, γ_M
- for total shear stress analysis, the characteristic undrained shear strength is to be divided by the material factor, γ_M

For soil resistance to axial and lateral loading of piles, material factors are to be applied to the characteristic resistance as described in K.6.2.1. Material factors to be used are specified in subsection K.6.2.1 and K.6.2.2 and K.6.3.2.

K.6.1.2 Soil investigation

Generally, the extent of site investigations and the choice of investigation methods are to take into account the type, size and importance of the structure, uniformity of soil, seabed conditions and the actual type of soil deposits. The area to be covered by site investigations is to account for positioning and installation tolerances. The soil investigation should give basis for a complete foundation design, comprising evaluations for piled foundation of

- On-bottom stability of unpiled structure
- Lateral pile resistance
- Axial pile resistance
- Pile/soil/structure interaction
- Pile driveability predictions

For skirted foundations, the soil investigation should give basis for evaluations of

- Foundation stability
- Soil/structure interaction
- Skirt penetration
- Settlements

Site investigations are to provide relevant information about the soil to a depth below which possible existence of weak formations will not influence the safety or performance of the structure.

Site investigations are normally to comprise of the following type of investigations:

- site geology survey including assessment of shallow gas.
- topography survey of the seabed
- geophysical investigations for correlation with borings and in-situ testing
- soil sampling with subsequent laboratory testing
- in-situ tests, e.g. cone penetration tests

The soil investigation should provide the following type of geotechnical data:

- data for soil classification and description
- shear strength data and deformation properties, as required for the type of analysis to be carried out
- in-situ stress conditions

Further details and requirements related to the soil investigation are given in Norsk Standard NS3481 /2/ and NORSOK G-001.

K.6.2 Piled foundation

K.6.2.1 Axial pile resistance

General

Soil resistance against axial pile loads is to be determined by one, or a combination of, the following methods:

- load testing of piles
- semi-empirical pile resistance formulae based on pile load test data.

The soil resistance in compression is to be taken as the sum of accumulated skin friction on the outer pile surface and resistance against pile tip. In case of open-ended pipe piles, the resistance of an internal soil plug is to be taken into account in the calculation of resistance against pile tip. The equivalent tip resistance is to be taken as the lower value of the plugged (gross) tip resistance or the sum of skin resistance of internal soil plug and the resistance against pile tip area. The soil plug may be replaced or reinforced by a grout plug or equivalent in order to achieve fully plugged tip resistance.

The submerged weight of the pile below mudline should be taken into account.

For piles in tension, no resistance from the soil below pile tip is to be accounted for when the tip is located in cohesionless soils.

Examples of detailed calculation procedures may be found in /3/, /4/, /5/ and /9/. The relevance of alternative methods should be evaluated related to actual design conditions. The chosen method should as far as possible have support in a data base which fits the actual design conditions related to soil conditions, type and dimensions of piles, method of installation, type of loading etc. When such an ideal fit is not available, a careful evaluation of important deviations between data base and design conditions should be performed and conservative modifications to selected methods should be made.

Effect of cyclic loading

Effects of repeated loading are to be included as far as possible. In evaluation of the degradation, the influence of flexibility of the piles and the anticipated load history is to be included.

Cohesive soils

For piles in mainly cohesive soils, the skin friction is to be taken equal to or smaller than the undrained shear strength of undisturbed clay within the actual layer. The degree of reduction depends on the nature and strength of clay, method of installation (e.g. driven or drilled/grouted), time effects, geometry and dimensions of pile, load history and other factors. Especially, the time required to achieve full consolidation after installation of driven piles should be considered. Design conditions that may occur prior to full consolidation should be checked for reduced resistances.

The unit tip resistance of piles in mainly cohesive soils may be taken as 9 times the undrained shear strength of the soil near the pile tip.

Cohesionless soil

For piles in mainly cohesionless soils the skin friction may be related to the effective normal stresses against the pile surface by an effective coefficient of friction between the soil and the pile element. Examples of recommended calculation methods and limiting skin friction values may be found in /3/. The calculation procedures given in /9/ for cohesionless soils should not be used before considerably more extensive documentation is available or without a careful evaluation of limiting values.

The unit tip resistance of piles in mainly cohesionless soils may be calculated by means of conventional bearing capacity theory, taking into account a limiting value which may be governing for long piles /3/.

Material factors

For determination of design soil resistance against axial pile loads in ULS design, a material factor, $\gamma_M=1.3$ is to be applied to all characteristic values of soil resistance, e.g. to skin friction and tip resistance.

For individual piles in a group lower material factors may be accepted, provided that the pile group as a whole is designed with the required material factor. A pile group in this context is not to include more piles than those supporting one specific leg.

Group effects should be accounted for when relevant, as further detailed in K.6.2.2.

K.6.2.2 Lateral pile resistance

When lateral soil resistance governs pile penetrations, the design resistance is to be checked within the limit state categories ULS and ALS, using following material factors applied to characteristic resistance:

$\gamma_M = 1.3$ for ULS condition

$\gamma_M = 1.0$ for ALS condition

For calculation of pile stresses and lateral pile displacements, the lateral soil reaction is to be modelled using characteristic soil strength parameters, with the soil material factor $\gamma_M=1.0$

The non-linear mobilisation of soil resistance is to be accounted for.

The effect of cyclic loading should be accounted for in the lateral load-deflection (p-y) curves.

Recommended procedures for calculation of p-y curves may be found in /3/, /4/.

Scour

The effect of local and global scour on the lateral resistance should be accounted for. The scour potential may be estimated by sediment transport studies, given the soil particle sizes, current velocity etc. However, experience from nearby similar structures, where available, may be the most important guide in defining the scour criteria.

Group effects

When piles are closely spaced in a group, the effect of overlapping stress zones on the total resistance of the soil is to be considered for axial as well as lateral loading of the piles. The increased displacements of the soil volume surrounding the piles due to pile-soil-pile interaction and the effects of these displacements on interaction between structure and pile foundation is to be considered.

In evaluation of pile group effects, due considerations should be given to factors as:

- pile spacing
- pile type
- soil strength and deformation properties
- soil density
- pile installation method.

K.6.2.3 Foundation response analysis

The pile responses should preferably be determined from an integrated pile/soil/structure analysis, accounting for the soils' non-linear response and ensuring load-deflection compatibility between the structure and the pile/soil system. Such an analysis is normally carried out with characteristic soil strength parameters.

K.6.2.4 Installation

General

The structure shall be documented to have adequate foundation stability after touchdown, as well as before and after piling, when subjected to the environmental and accidental actions relevant during this period.

Pile stress check prior to driving

The pile should be checked with respect to yield and buckling (ULS) in the maximum possible inclined position for a design condition of maximum relevant equipment weight (e.g. hammer weight), plus pile self-weight. Current loads and possible dynamic effects should be accounted for.

Pile driving

It should be demonstrated by calculations that the indented driving equipment is capable of driving the pile to target penetration within the pile refusal criteria specified and without damaging the piles.

The soil resistance during driving used in the driveability analysis may account for the gradual reduction in the skin friction caused by driving. The set-up effects leading to increased skin friction after stop of driving should be taken into account for realistic duration of halts that may occur during driving.

Dynamic stresses caused by pile driving are to be assessed based upon recognised criteria or by using wave equation analysis. The sum of the dynamic driving stresses and the static stresses during the driving process is not to exceed the specified minimum yield strength.

Allowance for under- or over-drive to account for uncertainties in pile-driving predictions should be considered.

K.6.2.5 Pile fatigue

The pile fatigue damage should be evaluated and demonstrated to be within the requirements. Considering the pile as a structural component with no inspection access and a substantial consequence of damage, a fatigue design factor of 10 should be applied, as further outlined in Section K.4.4.3

It should be noted that the pile fatigue damage from the pile driving operation normally contributes substantially to the total fatigue damage, which is the sum of the partial damages from the pile driving and the environmental action during the design service life.

Appropriate S-N-data for double-sided butt welds in piles is the D-curve.

The D-curve for air may be applied for the pile driving sequence.

The D-curve for seawater shall be used for the long term wave actions expected during the service life of the platform.

Provisions for use of the D-curve is that the weld be made in the shop and that all weld runs are made up with the pile in the horizontal position by use of submerged arc welding technology, and that 100% MPI is performed.

The fatigue design factor should be applied both to the pile driving damage and to the environmental damage.

K.6.2.6 Foundation simulation for jacket fatigue analysis

For a jacket fatigue analysis, the pile/soil system may be simulated by pile stiffness matrices generated from independent pile/soil analysis. To account for the soil non-linearity, the pile analysis for derivation of pile fatigue stiffness matrices should be performed at a representative fatigue load level.

K.6.3 Skirted foundation

K.6.3.1 General

Skirted foundations are an alternative to pile foundations for jacket structures. The base area can be of any shape and the skirts can be corrugated or plane. The skirts are penetrating the soil and contribute to increase the foundation capacity with respect to horizontal as well as vertical forces. Also, uplift may be resisted by the skirted foundation to a variable degree depending on the soil and loading conditions.

The skirted foundation may be used in both cohesive and cohesionless soils. The design for the main dimensions of the foundation should consider both the soil and the structural behaviour during operation as well as during installation. Possibility of inhomogeneity of the soil over the area and local seabed variations should be accounted for.

K.6.3.2 Foundation capacity

In calculation of the foundation capacity one should consider the simultaneously acting vertical and horizontal forces and overturning moment acting on the skirted foundation. The acting forces should be determined from a soil structure interaction analysis. The stiffnesses used to simulate the foundation should account for the non-linearity of the soil and be compatible with the load level of the soil reactions obtained from the analysis. This may require iterative analyses unless an adequate fully integrated analysis tool is used.

The determination of the foundation capacity should account for the fact that most soils, even sandy soils, behave undrained for the short duration of a wave action, and that the cyclic loading effects may have significant effect on the undrained strength. The determination of foundation capacity should preferably be based on cyclic strengths derived by combining relevant storm load histories for the loads on the skirted foundations with the load dependent cyclic strengths determined from undrained cyclic soil tests. Depending on the reliability of determining cyclic strengths for the soil in question the use of large-scale field tests should be evaluated.

Special concerns for clay

In clay the soil can normally be considered fully undrained for all the accumulated effects of a design storm, and cyclic strengths can be determined based on recognised principles, as e.g. given in /4/, and described in more detail in /7/. It is important that the cyclic strength is determined for a realistic relation between average and cyclic shear stresses, representative for the storm load history on the skirted foundation.

Special concerns for sand

Even in sand, the transient type of loading from one wave will result in an undrained or partly undrained response in the soil. If the sand is dense dilatancy effects would lead to high shear resistance for a single transient loading, resulting in high capacity for any type of loading, including uplift. However, when the soil is exposed to repeated loading in an undrained state, accumulated pore pressures will result. Such pore pressures can ultimately lead to a 'true failure' for less dense sands, or a state of 'initial liquefaction' or 'cyclic mobility' for dense sands. In the latter case, large cyclic soil strains and corresponding displacements of the foundation may occur before the soil dilates and higher resistance is obtained. It is recommended that the definition of failure is based on an evaluation of tolerable cyclic or total strains in the soil.

Even though the sandy soils will be undrained or partly undrained from the transient loads from each wave, there may be a considerable dissipation of excess pore pressures through the duration of a storm. The combined effects of pore pressure build up from cyclic loading and dissipation may be accounted for. It should be aimed at defining an equivalent load history with a limited duration (as compared to the duration of the storm) for which the soil can be assumed to be basically undrained. An example of a calculation procedure is described in /10/. This load history may be used in defining cyclic shear strengths of the soil for calculation of cyclic capacities, or scaled directly as loads onto large-scale tests if such tests are performed.

The general failure mechanisms for skirted foundations in sand are described in more detail in part 2 of /6/, together with description of the design approach used for a jacket on dense sand.

Material factors

The minimum values of the material factors should be those specified by NORSOK N-001, i.e. $\gamma_M=1.25$ for ULS design and $\gamma_M=1.0$ for ALS design.

There are elements in the design process for foundation stability design of a skirted foundation that may contribute to greater uncertainties than for other type of foundations. This especially relates to defining the cyclic strengths in sandy soils. Depending on the conservatism used in the design it may be necessary to increase the material factors generally recommended by NORSOK N-001. It is recommended that the sensitivity of the main assumptions made in the design process is investigated before such a decision is made.

K.6.3.3 Skirt penetration

For installation of the skirted foundations the following design aspects should be considered

- A sufficient penetrating force to overcome the soil resistance against penetration should be provided by a combination of weight and suction.
- The suction should be applied in a controlled manner to prevent excessive soil heave inside the skirted foundation
- The skirted foundation should be designed to prevent buckling due to the applied suction
- Channelling

Penetration resistance

In calculation of penetration resistance, there are several uncertainties related to

- spatial variations in soil conditions; may be different at different corners
- general uncertainties in determination of basic soil parameters
- uncertainties in calculation of penetration resistance from basic soil parameters

The total range of expected resistances should be documented. The upper bound resistance will be governing for design of ballast and/or suction system and for structural design of the foundation. Depending of the conservatism applied in choice of parameters and method of calculation to cover up for the uncertainties listed above, safety factors may have to be applied to the calculated resistances.

In clayey soils it can be considered that the resistance is unaffected by the applied suction, and the resistance can be calculated as for skirts penetrated by weight only. Skirt penetration resistances may be determined through correlations with cone penetration resistances as given in /4/, relevant basically for overconsolidated clays. Alternatively the skin friction may be related to the remoulded shear strength of the clay, and tip resistance may be related to the intact shear strength through conventional bearing capacity factors.

In sand the penetration resistance will be effectively reduced by applied suction. The reduction is caused by

- general decrease of effective stresses inside the skirted foundation reducing the skin friction
- large gradients in effective stresses combined with large inwards gradients in the flow below the skirt tip level, which triggers an inward bearing failure beneath the skirt tip and thus significantly reduces the otherwise in sand dominating tip resistance.

The general governing mechanisms are described in /7/ and /8/. The reduction effect of suction on penetration resistance is dependent both on the geometry of the skirted foundations and on the soil stratification. When penetrating sand layers that are overlain by a clay layer, significantly higher suction will be required than in homogeneous sand. Reliable universal calculation methods for calculation of penetration resistance in sand when suction is applied are not generally available. The estimation penetration resistance should thus to a high extent be based on empirical evidence from relevant conditions, taking account for the effects described above.

Control of soil heave

The control of soil heave should be based on a detailed monitoring of penetration, tilt, heave of soil plug, suction pressures, etc. As long as there is a net weight from the jacket acting on the skirted foundation, the amount of soil heave inside the skirted foundation is normally controlled by controlling the discharge of water through the pumps. The restraining effects on a jacket with 4 or more legs should be taken into account. By applying suction to one skirted foundation only, the weight from the jacket on that skirted foundation will decrease. If the weight from the jacket is reduced to zero, significant soil heave may occur. This should be avoided. A penetration procedure should normally be aimed at, where the jacket as far as possible is penetrated in a level position.

K.6.3.4 Skirted foundation structural design

Reference is made to Section 6 and 8 for general static strength and fatigue requirements. The skirted foundation should be designed for all relevant design conditions including

- transportation forces
- forces when lowering the skirted foundations into the water and through the water line
- forces due to the motions of the jacket when the skirts start penetrating the seabed
- the suction applied during skirt penetration
- operational forces

The buckling resistance during penetration of the skirts should be checked for various stages of the penetration using calculation models that account for the boundary conditions both at the skirt top as well as within the soil.

The design for operational loading conditions should be based on soil reaction distributions that are derived from realistic soil/structure interaction analyses. Alternatively more simple approaches with obviously conservative soil reaction distributions can be used.

K.6.4 On-bottom stability

The foundation system for the jacket temporary on-bottom condition prior to installation of the permanent foundation system shall be documented to have the required foundation stability for the governing environmental conditions as specified, and for all relevant limit states.

K.7 DOCUMENTATION REQUIREMENTS FOR THE ‘DESIGN BASIS’ & ‘DESIGN BRIEF’

K.7.1 General

Adequate planning shall be undertaken in the initial stages of the design process in order to obtain a workable and economic structural solution to perform the desired function. As an integral part of this planning, documentation shall be produced identifying design criteria and describing procedures to be adopted in the structural design of the platform.

Applicable codes, standards and regulations shall be identified at the commencement of the design.

When the design has been finalised, a summary document containing all relevant data from the design and fabrication and installation phase (DFI) shall be produced.

Design documentation (see below) shall, as far as practicable, be concise, non-voluminous, and, should include all relevant information for all relevant phases of the lifetime of the unit.

General requirements to documentation relevant for structural design are given in NORSOK, N-001, Section 5.

K.7.2 Design basis

A Design Basis Document shall be created in the initial stages of the design process to document the basis criteria to be applied in the structural design of the unit.

A summary of those items normally to be included in the Design Basis document is included below.

- Platform location and main functionalities,
- general description, main dimensions and water depth,
- applicable Regulations, Codes and Standards (including revisions and dates),
- service life of platform,
- topside interface requirements (including leg spacing, topside weight and c.o.g, appurtenance dimensions and routing),
- materials,
- coating and corrosion protection system,
- environmental and soil data,
- in-service inspection philosophy
- installation method,
- foundation system,
- system of units

K.7.3 Design brief

A Design Brief (DB) shall be created in the initial stages of the design process. The purpose of the Design Brief shall be to document the criteria and procedures to be adopted in the design of the jacket structure.

The Design Brief shall, as far as practicable, be concise, non-voluminous, and, should include all relevant limiting design criteria for the relevant design phase.

Design Briefs shall as a minimum cover the following main design phases and designs:

- Load-out
- Transportation
- Installation
- Inplace
- Fatigue
- Earthquake and accidental
- Foundation, on-bottom stability and pile driving

A summary of those items normally to be included in the Design Brief is included below.

Environmental design criteria

Limiting environmental design criteria (including all relevant parameters) for relevant conditions, including :

- wind, wave, current, earthquake, snow and ice description for relevant annual probability events.

Temporary phases

Design criteria for all relevant temporary phase conditions including, as relevant :

- limiting permanent, variable, environmental and deformation action criteria,
- essential design parameters and analytical procedures associated with temporary phases e.g. for load-out, transportation, lifting, installation, on-bottom stability and pile installation and driving,
- platform abandonment.

Operational design criteria

Design criteria for relevant operational phase conditions including :

- limiting permanent, variable, environmental and deformation action criteria,
- deck load description (maximum and minimum) including variation in gravity,
- designing accidental event criteria (e.g. collision criteria, earthquake, explosion and fire),
- soil parameters.

Global structural analyses

A general description of models to be utilised in the global analysis including :

- description of global analysis model(s) including modelling for wave and current loading,
- foundation system,
- description of analytical procedures (including methodology, factors, dynamic representation and relevant parameters).

Structural evaluation

A general description of the structural evaluation process including :

- description of local analytical models,
- description of procedures to be utilised for combining global and local responses,
- criteria for member and joint code checking,
- description of fatigue analytical procedures and criteria (including design fatigue factors, S-N-curves, basis for stress concentration factors (SCF's), etc.).

Miscellaneous

A general description of other essential design information, including :

- description of corrosion allowances, where applicable,
- in-service inspection criteria (as relevant for evaluating fatigue allowable cumulative damage ratios).

K.8 REFERENCES

- /1/ International Standard ISO 19900: 'Petroleum and natural gas industries – Offshore structures, General requirements'
- /2/ Norges Standardiseringsforbund, NS3481: 'Soil investigation and geotechnical design for marine structures', 1989
- /3/ American Petroleum Institute, API RP2A – LRFD: 'Planning, designing and constructing fixed offshore platforms – Load and Resistance Factor Design', 1993
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DESIGN OF STEEL STRUCTURES
ANNEX L
SPECIAL DESIGN PROVISIONS FOR SHIP SHAPED UNITS

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L.1 INTRODUCTION

L.1.1 General

This Annex is intended to provide requirements and guidance to the structural design of ship shaped units constructed in steel, according to the provisions of relevant NORSOK standards. The Annex is intended as the rest of the standard to fulfill NPD regulations relating to design and outfitting of facilities etc in the petroleum activities /4/ and NORSOK N-001. In addition it is intended that the unit shall fulfill technical requirements to standard ship hull design. Therefore references to the technical requirements in maritime standards such as NMD, IMO and DNV classification rules for guidance and requirements to design is also given.

The Annex is intended as being generally applicable to all types of conventional ship shaped structures, including the following variants:

- Floating Production Units (FPU)
- Floating Storage and Offloading Units (FSU)
- Floating Production, Storage and Offloading Units (FPSO)
- Floating Production, Drilling, Storage and Offloading Units (FPDSO).

In this Annex the above will collectively be referred to as “units”.

This Annex is intended to cover several variations with respect to conceptual solutions as listed below:

- Units intended for production which may be equipped with topside structures, supporting the production facilities.
- Units intended for storage with storage tanks together with facilities for offloading to shuttle tankers.
- Units intended for production will normally have a turret installed, while units intended for storage only, may have a buoy installed, replacing the turret.
- Units that can either be permanently moored on site or have a disconnectable mooring system. In the latter case, the unit may disconnect from its moorings and leave the site under its own power or assisted by tugs, to avoid certain exceptional events such as extreme storms, icebergs, hurricanes etc. Normally the mooring lines are connected to the turret or buoy, positioned forward of the midship area.

Requirements concerning mooring and riser systems other than the interfaces with the structure of the units are not explicitly considered in this Annex.

The intention of this Annex is to cover units weather vaning by rotating around a turret or a buoy. Fixed spread mooring arrangements (and similar) should be specially considered.

L.1.2 Definitions

Ship Shaped Floating Production Units:

A floating unit can be relocated, but is generally located on the same location for a prolonged period of time. Inspections and maintenance can be carried out on location. The Ship Shaped Floating Productions unit may consist of a ship shaped hull, with turret, and production equipment on the deck.

Ship Shape Floating Storage and Offloading Units:

A floating unit can be relocated, but is generally located on the same location for a prolonged period of time. Inspections and maintenance can be carried out on location. The Ship Shaped Floating Storage and Offloading units normally consist of a ship shaped hull equipped for crude oil storage. The crude oil may be transported to shore by shuttle tankers via an offloading arrangement.

Ship Shaped Floating Production, Storage and Offloading Units:

A floating unit can be relocated, but is generally located on the same location for a prolonged period of time. Inspections and maintenance are carried out on location. The Ship Shaped Floating Production, Storage and Offloading unit normally consists of a ship shaped hull, with turret, and production equipment on the deck. The unit is equipped for crude oil storage. The crude may be transported to shore by shuttle tankers via an offloading arrangement.

Ship Shaped Floating Production, Drilling, Storage and Offloading Units:

A floating unit can be relocated, but is generally located on the same location for a prolonged period of time. Inspections and maintenance are carried out on location. The Ship Shaped Floating Production, Drilling, Storage and Offloading unit normally consists of a ship shaped hull, with turret, and production and drilling equipment on the deck. The unit is equipped for crude oil storage. The crude may be transported to shore by shuttle tankers via an offloading arrangement.

Turret:

A device providing a connection point between the unit and the combined riser- and mooring- systems, allowing the unit to rotate around the turret (weather vane) without twisting the risers and mooring lines.

RCS:

Recognised Classification Society

L.2 BASIS OF DESIGN

L.2.1 Safety format

Generally, the design of units shall be based upon the partial factor design methodology as described in NORSOK N-001, based on direct calculated actions as described in L.5.

To ensure that experiences with traditional ships are taken into account in the design, units shall as a minimum, fulfil the relevant technical requirements given in the DNV Rules for Ship, Pt.3 Ch.1, /3/. Due consideration should be made with respect to the relevance of actions specified, taking into account possible increase in action level due to the differences in operation of the unit compared to normal trading ships.

L.2.2 Design criteria

The design of units should as a minimum, comply with the technical requirements given in the DNV Rules for Ships, Pt.3 Ch.1, /3/.

In cases where the unit is registered in a specific country (as though it was a merchant ship), the relevant requirements of the flag state authority shall additionally be complied with.

A unit may be designed to function in a number of operational modes, e.g. transit, operation and survival. Limiting design criteria for going from one mode to another shall be clearly established and documented when relevant. In these operational modes, the design criteria normally relate to consideration of the following items:

- Intact strength covering transit, temporary conditions (e.g. installation), extreme operating conditions and survival conditions (relevant only if normal operation of the unit is terminated when limiting weather conditions for the extreme operating condition are exceeded).
- Structural strength in damaged condition
- Compartmentation and stability

If it is intended to dry-dock the unit, the bottom structure shall be suitable strengthened to withstand such actions.

L.2.2.1 Non-operational phases

The structure shall be designed to resist relevant actions associated with conditions that may occur during all relevant stages of the life cycle of the unit. Such stages may include:

- fabrication,
- site moves,
- sea transportation,
- installation, and,
- decommissioning.

Structural design covering marine operation construction sequences shall be undertaken in accordance with NORSOK, N-001.

Marine operations should be undertaken in accordance with the requirements stated in the VMO standard.

Sea Transportation

A detailed transportation assessment shall be undertaken which includes determination of the limiting environmental criteria, evaluation of intact and damage stability characteristics, motion response of the global system and the resulting, induced actions. The occurrence of slamming actions on the structure and the effects of fatigue during transport phases shall be evaluated when relevant.

Satisfactory compartmentation and stability during all floating operations shall be ensured.

Technical requirements to structural strength and stability as stated in relevant parts of the DNV Rules for Ships, /3/, e.g. Pt.3 Ch.1 and Ch. 4 should be complied with for transit conditions where such rules are found to be applicable.

Installation

Installation procedures of foundations (e.g. drag embedded anchors, piles, suction anchor or dead weights etc.) shall consider relevant static and dynamic actions, including consideration of the maximum environmental conditions expected for the operations.

Installation operations shall consider compartmentation and stability, and, dynamic actions on the mooring system(s). The actions induced by the marine environment involved in the operations and the forces exerted by the positioning equipment, such as fairleads and padeyes, shall be considered for local strength checks.

Decommissioning

Abandonment of the unit shall be planned for at the design stage. However, decommissioning phases for ship shaped units are normally not considered to provide design action conditions for the unit and may normally be disregarded at the design phase.

L.2.2.2 Operational phases

The unit shall be designed to resist all relevant actions associated with the operational phases of the unit. Such actions include ;

- Permanent actions (see L.4.2)
- Variable actions (see L.4.3)
- Deformation actions (see L.4.4)
- Environmental actions (see L.4.5)
- Accidental action (see L.4.6)
- Fatigue actions (see L.4.7)

Each mode of operation, such as production, work over, or, combinations thereof, shall be considered.

The unit may be designed to function in a number of operational modes, e.g. operational and survival.

Limiting design criteria for going from one mode of operation to another mode of operation shall be clearly established and documented.

L.3 STRUCTURAL CLASSIFICATION AND MATERIAL SELECTION

L.3.1 Structural classification

Selection of steel quality, and requirements for inspection of welds, shall be based on a systematic classification of welded joints according to the structural significance and the complexity of the joints/connections as documented in Chapter 5.

In addition to in-service operational phases, consideration shall be given to structural members and details utilised for temporary conditions, e.g. fabrication, lifting arrangements, towing arrangements, etc.

Basic Considerations

Structural connections in ship shaped units designed in accordance with this Annex will normally not fall within the categorisation criteria relevant for Design Class DC1 or DC2. In particular, relevant failure of a single weld, or element, should not lead to a situation where the accidental limit state damaged condition is not satisfied. Structural connections will therefore be categorised within classification groups DC3, DC4 or DC5.

Consideration shall be given to address areas where through thickness tensile properties may be required.

Special consideration shall be given to ensure the appropriate inspection category for welds with high utilisation in fatigue if the coverage with standard local area allocation is insufficient.

Examples of typical design classes applicable to ship shaped units are stated below. These examples provide minimum requirements and are not intended to restrict the designer in applying more stringent requirements should such requirements be desirable.

Typical locations - Design class : DC3

- Structure in way of moonpool
- Structure in way of turret, fairlead and winches supporting structure
- Highly stressed areas in way of main supporting structures of heavy substructures and equipment e.g. anchor line fairleads, cranes substructure, gantry, topside support stools, flare boom, helicopter deck, davits, hawser brackets for shuttle tanker, towing brackets etc.

DC3 areas may be limited to local, highly stresses areas if the stress gradient at such connections is large.

Typical locations - Design class : DC4

In general all main loadbearing structural elements not described as DC3, such as:

- Longitudinal structure
- Transverse bulkheads
- Transverse frames and stringers
- Turret structure
- Accommodation

Typical locations - Design class : DC5

Non main loadbearing structural elements, such as:

- Non-watertight bulkheads,
- Tween decks
- Internal outfitting structure in general
- other non-loadbearing components

L.3.2 Material selection

Material specifications shall be established for all structural materials utilised in a ship shaped unit. Such materials shall be suitable for their intended purpose and have adequate properties in all relevant design conditions. Material selection shall be undertaken in accordance with the principles given in NORSOK M-001.

When considering criteria appropriate to material grade selection, adequate consideration shall be given to all relevant phases in the life cycle of the unit. In this connection there may be conditions and criteria, other than those from the in-service, operational phase, that provide the design requirements in respect to the

selection of material. (Such criteria may, for example, be design temperature and/or stress levels during marine operations.)

Selection of steel quality for structural components shall normally be based on the most stringent Design Class of the joints involving the component.

Requirements to through-thickness strength shall be assessed.

The evaluation of structural resistance shall include relevant account of variations in material properties for the selected material grade. (e.g. Variation in yield stress as a function of thickness of the base material).

L.3.3 Inspection categories

Welding, and the extent of non-destructive examination during fabrication, shall be in accordance with the requirements stipulated for the appropriate 'inspection category' as defined in NORSOK, M-101.

The lower the extent of NDT selected, the more important is the representativeness of the NDT selected and performed. The designer should therefore exercise good engineering judgement in indicating priorities of locations for NDT, where variation of utility along welds is significant.

L.3.4 Guidance to minimum requirements

The following figures illustrate minimum requirements to selection of Design Class and Inspection Category for typical structural configurations of ship shaped units. The indicated Design Class and Inspection Categories should be regarded as guidance of how to apply the recommendations in chapter 5.

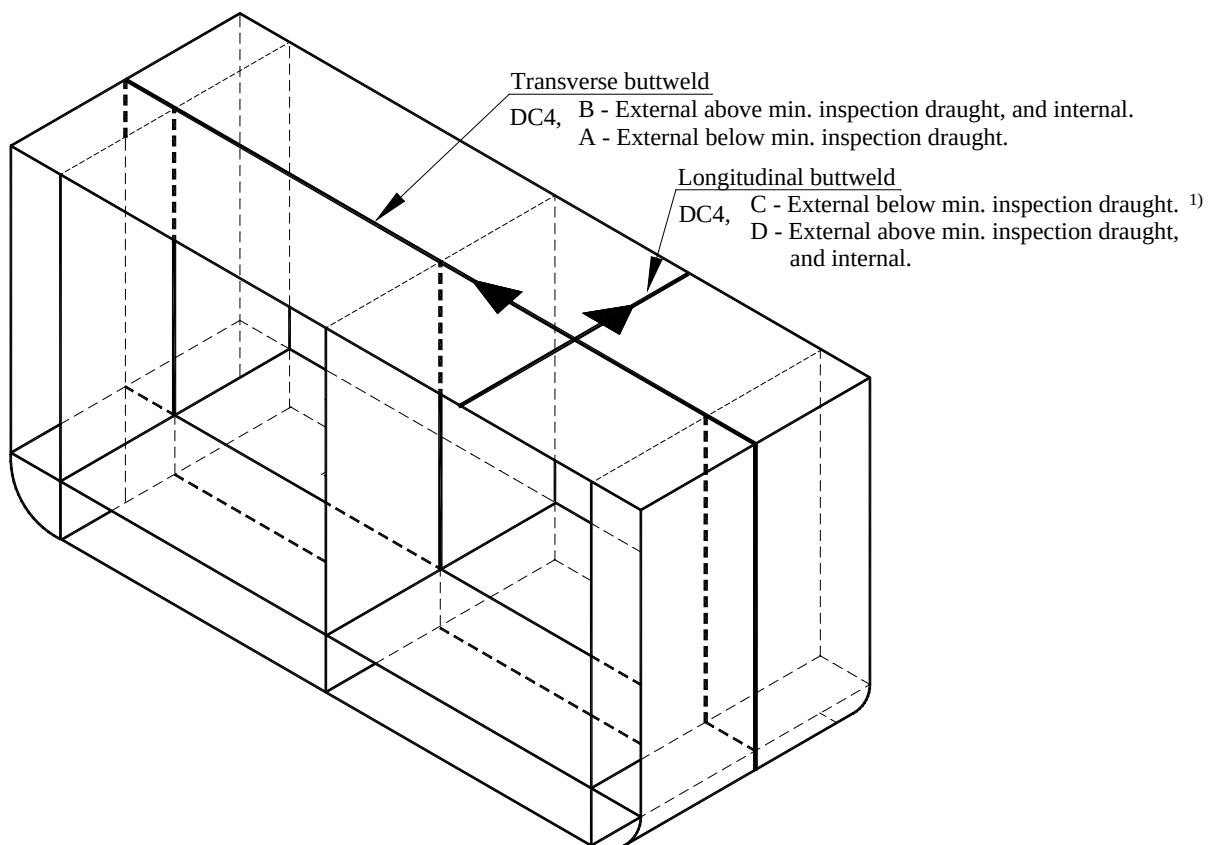


Figure L.3-1 Example of typical design classes and inspection categories for butt welds in the midship area

Key:

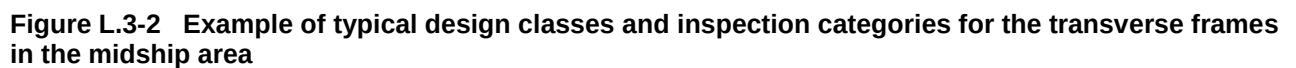
1. [DCn, ..] Design Class, n
2. [....., N] Inspection Category, N

Notes:

1. Inspection draught see L.7.3

2. The selection of the inspection categories is made on the following assumptions with reference to Table 5.3 and 5.4:

Transverse butt welds, external above min. insp. draught and internal stress Transverse butt welds, below min. insp. draught stress for Longitudinal butt welds, external above min insp. draught and internal Longitudinal butt welds, below min. insp. draught for	High fatigue, (ref. Table 5.4), dominating dynamic principal transverse to the weld. High fatigue, (ref. Table 5.4), dominating dynamic principal transverse to the weld. No access in-service inspection and repair. Low fatigue, low tensile stress transverse to the weld. Low fatigue, low tensile stress, transverse to the weld. No access in-service inspection and repair.
--	--



1. [DCn, ..] Design Class, n
2. [....., N] Inspection Category, N

1. Inspection draft see L.7.3
2. The Design Classes and Inspection Categories should be applied along the whole length of the welds
3. For welds of longitudinal girders and stringers MPI should be mandatory in way of transverse frame intersection.

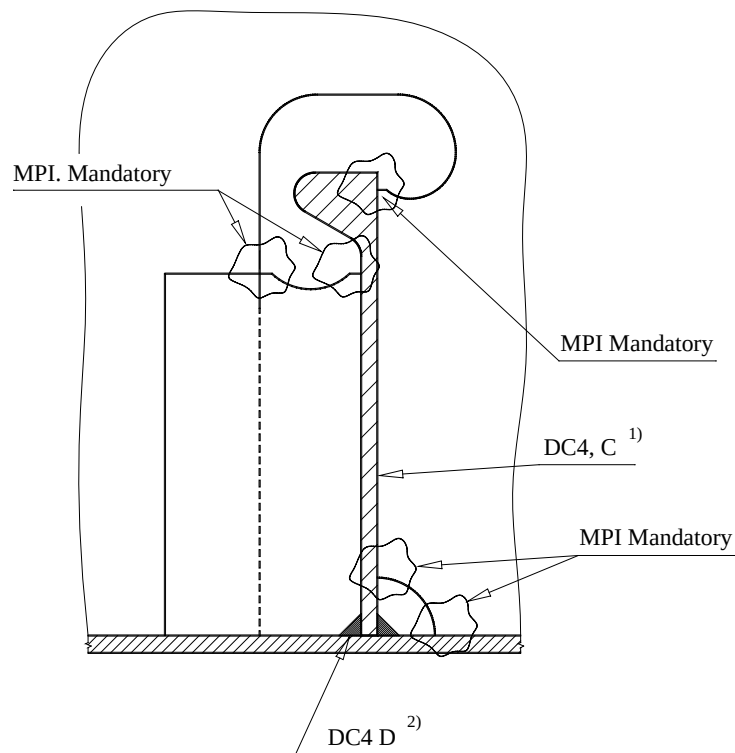


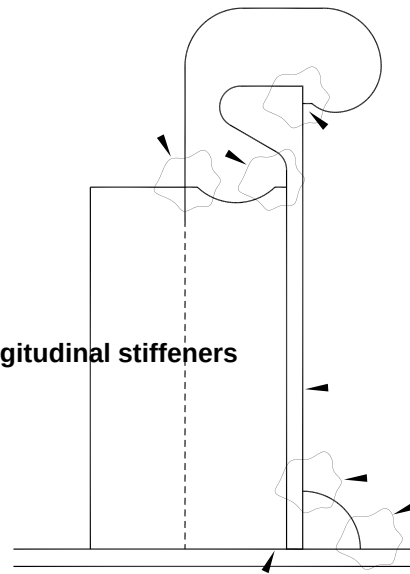
Figure L.3-3 Example of typical design classes and inspection categories longitudinal stiffeners connection to transverse frames / bulkheads

Key:

1. [DCn, ..] Design Class, n
2. [....., N] Inspection Category, N

Notes:

1. The selected inspection class is relevant for a high fatigue loading with principal stresses parallel to the weld and assuming in-service access (see table 5.4)
2. The selected inspection class is relevant for a low fatigue joint with low tensile stresses perpendicular to the weld



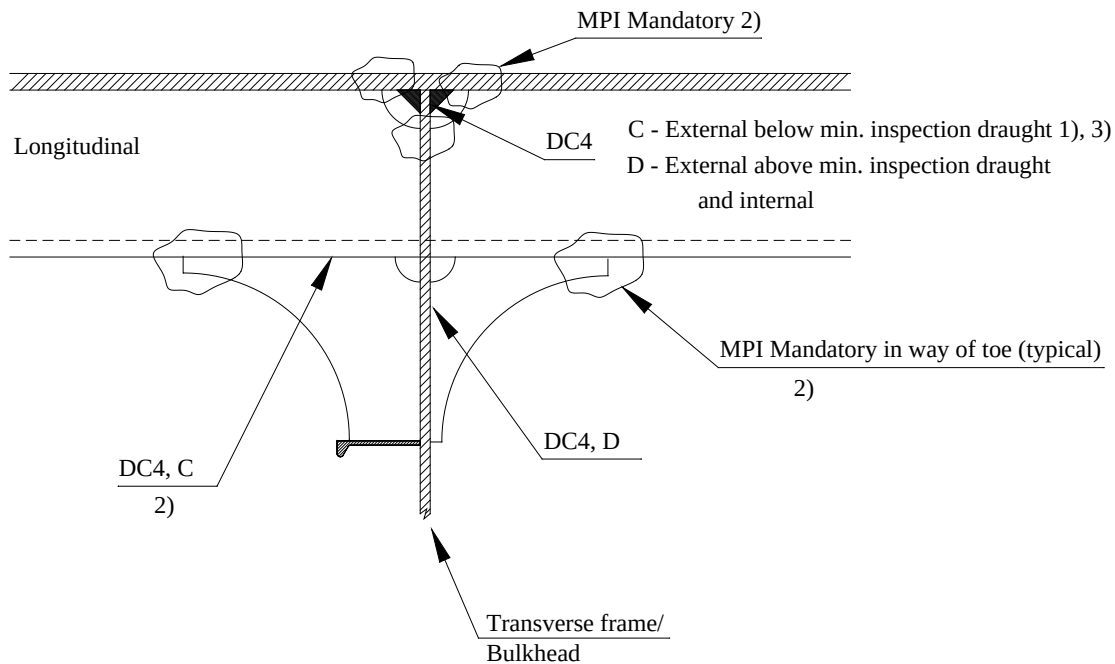


Figure L.3-4 Example of design classes and inspection categories longitudinal stiffeners connection to transverse frames / bulkheads

Key:

1. [DCn, ..] Design Class, n
2. [..., N] Inspection Category, N

Notes:

1. Inspection draft see L.7.3
2. The proposed inspection categories are relevant for high fatigue loading, for longitudinal stiffener in the sidshele.
3. The proposed inspection categories are relevant for low fatigue loading with low tensile stresses transverse to the weld, and no access for repair below min. inspection draught.

L.4 DESIGN ACTIONS

L.4.1 General

Characteristic actions shall be used as reference actions in the partial coefficient method.

Design actions relevant for ship shaped units are in general defined in NORSOK N-003. Guidance concerning action categories relevant for ship shaped unit designs are given in the following.

Design action criteria dictated by operational requirements shall be fully considered. Examples of such requirements may be:

- drilling (if applicable), production, workover and combination thereof,
- consumable re-supply procedures and frequency,
- maintenance procedures and frequency,
- possible action changes in extreme conditions.

L.4.2 Permanent actions (G)

Permanent actions are actions that will not vary in magnitude, position or direction during the period considered. Permanent actions relevant for ship shaped unit designs are:

- 'lightmass' of the unit including mass of permanently installed modules and equipment, such as accommodation, helideck, cranes, drilling (if applicable) and production equipment,
- hydrostatic pressures resulting from buoyancy,
- pretension in respect to mooring, drilling (if applicable) and production systems (e.g. mooring lines, risers etc.).

L.4.3 Variable actions (Q)

Variable actions are actions that may vary in magnitude, position and direction during the period under consideration.

Except where analytical procedures or design specifications otherwise require, the value of the variable actions utilised in structural design should normally be taken as either the lower or upper design value, whichever gives the more unfavourable effect. Variable actions on deck areas for local, primary and global design are stated in NORSOK, N-003. For global design the factor to be applied to primary design actions (as stated NORSOK, N-001) is not, however, intended to limit the variable load carrying capacity of the unit in respect to non-relevant, variable load combinations. Global load cases should be established based upon 'worst case', representative variable load combinations, where the limiting global criteria is established by compliance with the requirements to intact and damage hydrostatic and hydrodynamic stability.

Global design action conditions, and associated limiting criteria, shall be documented in the stability manual for the unit, see L.9.

Variations in operational mass distributions (including variations in tank filling conditions) shall be adequately accounted for in the structural design.

Local design actions for all decks shall be documented on a 'load-plan'. This plan shall clearly show the design uniform and concentrated actions for all deck areas for each relevant mode of operation.

Dynamic actions resulting from flow through air pipes during filling operations shall be adequately considered in the design of tank structures.

L.4.4 Deformation actions (D)

Deformation actions are actions due to deformations applied to the structure.

Deformation actions resulting from construction procedures and sequences (e.g. as a result of welding sequences, forced alignments of structure etc.) shall be accounted for when relevant.

Other relevant deformation action effects may include those resulting from temperature gradients, as for example:

- when hot-oil is stored in the cargo tanks,
- radiation from flaring operations.

Deflection imparted to topside structure due to G, Q or EA actions acting on the ship hull, shall not be considered as deformation actions.

L.4.5 Environmental actions (EA)

Environmental actions are actions caused by environmental phenomena. The characteristic value of an environmental action is the maximum or minimum value (whichever is the most unfavourable) corresponding to an action effect with a prescribed probability of exceedance.

Environmental conditions shall be described using relevant data for the relevant period and areas in which the ship shaped unit is to be fabricated, transported, installed and operated.

The long-term variation of environmental phenomena such as wind, waves and current shall be described by recognised statistical distributions relevant to the environmental parameter considered, (NORSOK N-003). Information on the joint probability of the various environmental actions may be taken into account if such information is available and can be adequately documented.

Consideration shall be given to responses resulting from the following listed environmental induced action effects that may be relevant for the design of a ship shaped unit:

- wave actions acting on the hull, (including variable sea pressure)
- wind actions,
- current actions,
- temperature actions,
- snow and ice actions.
- dynamic stresses for all limit states (e.g. with respect to fatigue see L.4.7)
- rigid body motion (e.g. in respect to air gap and maximum angles of inclination),
- sloshing,
- slamming (e.g. on bow and bottom in fore and aft ship)
- green water on the deck
- vortex induced vibrations (e.g. resulting from wind actions on structural elements in a flare tower),
- wear resulting from environmental actions at mooring and riser system interfaces with hull structures,

For ship shaped units with traditional, catenary mooring systems, earthquake actions may normally be ignored.

Further considerations in respect to environmental actions are given in NORSOK, N-003.

Analytical considerations with regard to evaluation of global response resulting from environmental action components are stated in L.5.

L.4.5.1 Mooring actions

A unit may be kept on station by various methods, depending on specific site criteria and operational goals. These methods may include several different types of station-keeping systems such as internal and submerged turret systems, external turret, CALM buoy, fixed spread mooring and dynamic positioning. Each mooring system configuration will impose actions into the hull structure which are characteristic to that system. These actions shall be addressed in the structural design of the unit, and combined with other relevant action components. Only mooring arrangements, which allows weather vaning of the unit, are covered by this annex.

L.4.5.2 Sloshing actions in tanks

Merchant ships are normally operated so that cargo tanks are typically either nearly full or nearly empty. For a floating production unit, in general, no restrictions should be imposed on partly filling of cargo tanks, and some cargo tanks may be partially filled most of the time. For partially filled tanks, the phenomenon of resonant liquid motion (sloshing) inside the tank should be considered.

Sloshing is defined as a dynamic magnification of internal pressures acting on the boundaries of cargo tanks and on internal structure within the tank, to a level greater than that obtained from static considerations alone. Sloshing occurs if the natural periods of the fluid and the vessel motions are close to each other. Major factors governing the occurrence of sloshing are:

- tank dimensions
- tank filling levels
- structural arrangements inside the tank (wash bulkheads, web frames etc.)
- transverse and longitudinal metacentric height (GM)
- vessel draught
- natural periods of ship and cargo in roll (transverse) and pitch (longitudinal) modes.

The pressure fields created by sloshing of the cargo/ballast may be considered, according to the requirements given in DNV-OS-C102 /12/. The DNV Rules differentiate between ordinary sloshing loads (non-impact) and sloshing impact actions. To arrive at 100 year return period impact pressures, the 10^{-8} values should be multiplied by a factor 1.15 (See Commentary).

For units to be operated in severe environments, direct calculation of sloshing pressures using site-specific criteria, should be considered.

L.4.5.3 Green water effect

The green water effect is the overtopping by water in severe wave conditions. Significant amounts of green water will influence the deck structural design, accommodation superstructure, equipment design and layout, and may induce vibrations in the hull. Normally the forward part of the deck and areas aft of midship will be most severe exposed to green water. Short wave periods are normally most critical.

Appropriate measures should be considered to avoid or minimize green water effects on the ship structure and topside equipment. These measures include bow shape design, bow flare, breakwaters and other protective structure. Adequate drainage arrangements shall be provided.

Exposed structural members or topside equipment on the weather deck shall be designed to withstand the loads induced by green water. For (horizontal) weather deck structural members this load will mainly be the hydrostatic load from the green water. For horizontal or vertical topside members this load will mainly be the dynamic action from the green water rushing over the deck.

In lack of more exact information, for example from model testing, relevant technical requirements of the DNV-OS-C102 /12/ may be applied.

L.4.5.4 Slamming in the fore and aft ship

Slamming is impact from waves which give rise to an impulsive pressure from the water, resulting in a dynamic transient action (whipping) of the hull. The most important locations to be considered with respect to slamming are the forward bottom structure, the bow flare and accommodation structure in the fore ship. Slamming forces should be taken into account in the turret design when relevant. Other locations on the hull e.g. exposed parts of the aft ship, which may be subject to wave impacts should be considered in each separate case.

The frequency of occurrence and severity of slamming are significantly influenced by the following:

- vessel draught,
- hull geometrical form,
- site environment,
- heading,
- forward speed (including current)
- position of superstructure.

The effects of slamming on the structure shall be considered in design particularly with regard to enhancement of global hull girder bending moments and shear forces induced by slamming, local strength aspects and limitations to ballast draft conditions.

In lack of more exact information, for example from model testing, relevant requirements of the DNV Rules for Classification of Ships, /3/, may be applied:

- for bottom slamming in the ship fore body, Part 3, Chapter 1, Section 6, Bottom Structures, H200, Strengthening against slamming
- for bow flare slamming in the ship fore body, Part 3, Chapter 1, Section 7, Side Structures, E100, Strengthening against bow impact.
- for bottom slamming in the ship aft body, Part 3, Chapter 1, Section 7, Side Structures, E200, Stern slamming

For bottom slamming in the ship fore body, in order to account for:

- 100 year return period
- Main wave direction head waves
- Long crested waves

the expression for coefficient c_2 in Part 3, Chapter 1, Section 6, Bottom Structures, H200, Strengthening against slamming, should be changed to:

$$c_2 = 1675 \left(1 - \frac{12T_{BF}}{L} \right)$$

For bow flare slamming, in order to account for:

- 100 year return period
- Main wave direction head waves
- Long crested waves

the expression for bow flare slamming pressures in the ship fore body, Part 3, Chapter 1, Section 7, Side Structures, E103, are to be multiplied by a factor:

$$fac = \left(1 + \frac{25}{L} \right)^2$$

The applicable speed V is not to be taken less than 8 knots.

For bottom slamming in the ship aft body, in order to account for:

- 100 year return period
- Main wave direction head waves
- Long crested waves

the expression for stern slamming pressure in Part 3, Chapter 1, Section 7, Side Structures, E203, are to be multiplied by a factor:

$$fac = \left(1 + \frac{25}{L} \right)^2$$

The resulting slamming impact pressures or bow impact pressures shall be multiplied by a factor 0.375 if they are to be applied as a mean pressure over a larger area (applicable for global structural evaluation).

L.4.6 Accidental actions (A)

Accidental actions are actions related to abnormal operation or technical failure.

In the design phase particular attention should be given to layout and arrangements of facilities and equipment in order to minimise the adverse effects of accidental events.

Risk analyses shall be undertaken to identify and assess accidental events that may occur and the consequences of such events. Structural design criteria for the ALS condition are identified from the risk analyses. Generally, the following ALS events are those required to be considered in respect to the structural design of a ship shaped unit :

- dropped objects (e.g. from crane handling),
- fire,
- explosion,
- collision,
- unintended flooding and counterflooding situations, and,
- abnormal wave events.

The counterflooding situations shall consider the possible loading conditions after an unintended flooding has occurred, to compensate for the heeling resulting from such flooding.

Further considerations in respect to accidental actions are given in Chapters 9 and Annex A.

L.4.7 Fatigue actions (F)

Repetitive actions, which may lead to possible significant fatigue damage, shall be evaluated. In respect to a ship shaped unit, the following listed sources of cyclic response shall, where relevant, be considered:

- waves (including those actions caused by dynamic pressure, sloshing / slamming and variable buoyancy),
- wind (especially when vortex induced vibration may occur),
- mechanical vibration (e.g. caused by operation of machinery),

- crane actions,
- full / empty variation of filling level in cargo tanks (low cycle).

The effects of both local and global dynamic response shall be properly accounted for when determining response distributions of repetitive action effects.

L.4.8 Combination of actions

Action coefficients and action combinations for the design limit states are, in general, given in NORSOK, N-003.

Structural strength shall be evaluated considering all relevant, realistic action conditions and combinations. Scantlings shall be determined on the basis of criteria that combine, in a rational manner, the effects of relevant global and local responses for each individual structural element.

A sufficient number of load conditions shall be evaluated to ensure that the most probable largest (or smallest) response, for the appropriate return period, has been established. (For example, maximum global, characteristic responses for a ship shaped unit may occur in environmental conditions that are not associated with the characteristic, largest, wave height. In such cases, wave period and associated wave steepness parameters are more likely to be governing factors in the determination of maximum/minimum responses.)

L.5 STRUCTURAL RESPONSE

L.5.1 General

The structure shall as a minimum comply with the technical requirements of the DNV Rules for Ship, /3/, Pt.3 Ch.1, see L.2.1, as applicable to a normally trading ship. In addition, the dynamic actions to which the hull may be subjected will vary from those associated with seagoing trading ships, and will require assessment. This assessment may be based on results of model testing and/or by suitable direct calculation methods of the actual wave actions on the hull at the service location, taking into account relevant service related factors. When the design is based on direct calculations, the software used should have been verified by model testing.

Sufficient action conditions for all anticipated pre-service and in-service conditions shall be determined and analysed to evaluate the most unfavourable design cases for the hull girder global and local strength analysis.

For units intended for multi field developments the most onerous still water and site specific environmental actions shall be considered for the design. The action conditions shall be stated in the Operating Manual, see L.10.6.

The units shall comply with requirements taking into account site specific service. Factors which may influence the hull actions and shall be accounted for, include the following:

- Site specific environmental conditions
- Effect of mooring system
- Long term service at a fixed location
- Seas approaching predominantly from a narrow sector ahead
- Zero ship speed
- Range of operating action conditions
- Tank inspection requirements
- Different return period requirements compared with normal trading tankers

Site specific environment; for ship shaped units of normal form, strength standards set by RCS are based on criteria relating to a world-wide trading pattern.

Effect of mooring system; static and dynamic mooring and riser actions can be substantial. Their effects on the hull girder longitudinal bending moments and shear forces shall be accounted for in the design calculations.

Long term service at a fixed location; seagoing ships would generally spend a proportion of their time in sheltered water conditions. Permanently moored units would normally remain on station all the time and disconnectable units would only move off station in certain conditions and would generally remain in the local area. In addition the expectation of the field life may be in excess of 20 years. This shall be accounted for e.g. in the design of corrosion protection systems.

Seas approaching from a narrow sector ahead; for seagoing ships in severe weather, steps would generally be taken to minimise the effects of such conditions such as altering course or alternative routing. Moored production/storage units cannot take avoiding action and will weathervane due to the combined effects of waves, wind and current, resulting in a greater proportion of waves approaching from bow sector directions.

Zero ship speed; the effect of forward ship speed would enhance static predictions of hull bending moments and shear forces and other factors such as slamming. Due to zero forward speed this enhancement would not apply to moored production/storage units.

Range of operating action conditions; ocean going tankers traditionally have a fairly limited range of operational action conditions and would typically be “fully” loaded or in ballast. Due to their oil storage capability moored units however potentially have an almost unlimited range of action conditions. These effectively cover a full spectrum of cases from ballast through intermediate conditions to fully loaded returning to ballast via offloading.

Tank inspection requirements; ocean-going ships would generally be taken to dry dock for periodic survey and repair. Moored units can be inspected on station. Thus a full range of conditions covering each tank empty in turn, and tank sections empty should be addressed as appropriate for necessary access for inspection and maintenance. These conditions should be combined with appropriate site-specific environmental actions.

Different return period requirements compared with normal trading tankers: normal ship class rule requirements are based on providing adequate safety margins based on a 20 years return period. This standard specifies that the design shall be based on data having a return period of 100 years for the site specific action conditions.

Taking into consideration the conditions listed above, calculations shall be carried out to address all design action conditions including fully loaded, intermediate operating conditions, minimum loaded condition, inspection conditions with each tank empty in turn, etc. Still water bending moment and shear force distributions shall be calculated for each case and stated in the Operating Manual, see L.10.6.

L.5.2 Analysis models

L.5.2.1 General

Analysis models relevant for in-place ULS and FLS are described in this chapter.

The finite element modelling of the hull structure should be carried out according with principles given in the following. Four typical modelling levels are described below. Other equivalent modelling procedures may also be applied.

L.5.2.2 Global structural model (Model level 1)

In this model a relatively coarse mesh extending over the entire hull should be used. The overall stiffness of the primary members of the hull shall be reflected in the model. The mesh should be fine enough to satisfactorily take account of the following:

- Vertical hull girder bending including shear lag effects
- Vertical shear distribution between ship side and bulkheads
- Stress distribution around large penetrations such as the moonpool
- Horizontal hull girder bending including shear lag effects
- Termination of longitudinal members

The models should be used for analysing global wave response and still water response where found relevant.

L.5.2.3 Cargo tank model (Model level 2)

The cargo tank analysis shall be used to analyse local response of the primary hull structural members in the cargo area, due to relevant internal and external action combinations. The extent of the structural model shall be decided considering structural arrangements and action conditions.

The mesh fineness shall be decided based on the method of action application in the model. The model should normally include plating, stiffeners, girders, stringers, web-frames and major brackets. For units with topside the stiffness should be considered in the tank modelling. In some cases the topside structure should be included in the model.

The cargo tank model may be included in the global structural model, see L.5.2.2.

The finite element model should normally cover the considered tank, and one half tank outside each end of the considered tank. The effect of non-structural elements in the topside may introduce additional stiffness and should be considered.

Conditions of symmetry should as far as possible, be applied at each end of the finite element model. The model should be supported vertically by distributed springs at the intersections of the transverse bulkheads with ship sides and longitudinal bulkheads. The spring constants shall be calculated for each longitudinal bulkhead and ship sides based on actual bending and shear stiffness and for a model length of three tanks. If horizontal unbalanced loads are applied to the model in transverse direction e.g. heeling conditions, horizontal spring supports should be applied at the intersections of all continuous horizontal structural members.

The following basic actions are normally to be considered for the ULS condition:

- Cargo and ballast loading, static and dynamic
- External sea pressure, static and dynamic
- Topside loading, vertical static and dynamic and also horizontal acceleration actions

These ULS responses are to be combined with global stresses, see L.5.2.2.

The following basic actions are normally to be analysed in this model for the FLS condition:

- Dynamic cargo and ballast actions

- External sea pressure range
- Topside loading, vertical dynamic and also horizontal acceleration actions

These FLS responses are to be combined with global stresses, see L.5.2.2.

L.5.2.4 Turret analysis (Model level 2)

A finite element model should be made of the turret structure. The model should reflect the geometry and stiffness of the structure. A model with relatively coarse mesh may be used.

In cases where the stress distribution in the moonpool is dependent of the relative stiffness between unit hull and turret a model comprising both the unit hull and the turret should be made.

Modelling of interface areas may require finer mesh and use of gap elements etc. to describe the actual stress flow.

Following actions should normally be considered:

- Static and dynamic actions from the turret itself
- Mooring actions
- Riser actions

The analysis should cover both the ULS and the FLS condition.

L.5.2.5 Local structural analysis (Model level 3)

The purpose of the local structural analysis is to analyse laterally loaded local stiffeners (including brackets) subject to large relative deformations between girders.

The following typical areas should be given particular attention:

- Longitudinal stiffeners between transverse bulkhead and the first frame at each side of the bulkhead
- Vertical stiffeners at transverse bulkheads with horizontal stringers in way of inner bottom and deck connection
- Horizontal stiffeners at transverse bulkheads with vertical stringers in way of inner side and longitudinal bulkhead connection
- Corrugated bulkhead connections

Local structural models may be included in the cargo tank analysis or run separately with prescribed boundary deformations or boundary forces from the tank model.

L.5.2.6 Stress concentration models (Model level 4)

For fatigue assessment, fine element mesh models shall be made for critical stress concentration details, for details not sufficiently covered by stress concentration factors given in recognised standards, see Annex C.

In some cases detailed element mesh models may be necessary for ultimate limit state assessment in order to check maximum peak stresses and the possibility of repeated yielding.

The following typical areas may have to be considered:

- Hopper knuckles in way of web frames
- Topside support stools
- Details in way of the moonpool
- Other large penetrations in longitudinal loadbearing elements
- Longitudinal bulkhead terminations
- Stiffener terminations
- Other transition areas when large change in stiffness occur

The size of the model should be of such extent that the calculated stresses in the hot spots are not significantly affected by the assumptions made for the boundary conditions.

Element size for stress concentration analyses is to be in the order of the plate thickness. Normally, shell elements may be used for the analysis. The correlation between different actions such as global bending, external and internal fluid pressure and acceleration of the topside should be considered in the fatigue assessment. For further details reference is made to DNV Classification Note 30.7, /11/.

The correlation between different actions such as global bending, external and internal fluid pressure and acceleration of the topside should be considered in the fatigue assessment.

L.5.3 Calculation of wave induced actions

According to Recognised Classification Society Rules, normal trading ships are designed according to the following environmental criteria:

- North Atlantic wave condition, 20 year return period
- Short crested sea
- Same probability for all wave directions relative to the ship

Specific requirements for weather vaning units covered by this Annex are given below.

L.5.3.1 General

Global linear wave induced actions such as bending moments and shear forces should be calculated by using either strip theory or three dimensional sink source (diffraction) formulation. Strip theory is a slender body theory and is not recommend when the length over beam ratio is less than 3.

Generally, the most importance global responses are midship vertical bending moments and vertical shear forces in the fore and aft body of the unit and the associated vertical bending moments. These responses should be calculated for head sea conditions. Horizontal and torsional moments may be of interest depending on the structural design, alone and in combination with other action components.

The calculation of wave induced actions may follow the following steps:

- Calculation of the relevant transfer functions (RAOs)
- Calculation of the 100 year linear response
- Evaluation of non-linear effects

When a 3-D diffraction program is used, the hydrodynamic model shall consist of sufficient number of facets. In general, the facets should be sufficiently modelled to describe the unit in a propitiate way. The size of the facets shall be determined with due consideration to the shortest wave length included in the hydrodynamic analysis. Smaller facets should be used in way of the water surface.

The mass model shall be made sufficiently detailed to give centre of gravity, roll radius of inertia and mass distribution as correct as practically possible.

L.5.3.2 Transfer functions

The following wave induced linear responses should be calculated:

- Motions in six degrees of freedom.
- Vertical bending moment at a sufficient number of positions along the hull. The positions have to include the areas where the maximum vertical bending moment and shear force occur and at the turret position. The vertical wave induced bending moment shall be calculated with respect to the section's neutral axis.
- Horizontal and torsional moment if applicable.

The linear transfer functions shall be calculated by either strip or 3D sink –source (diffraction) theory. The responses shall be calculated for a sufficient number of wave periods in the range from 4 – 35 seconds. For short crested sea at least seven directions in a sector $\pm 90^\circ$ relative to head sea, with maximum 30° interval, should be considered.

L.5.3.3 Viscous damping

If roll resonance occurs within the range of wave periods likely to be encountered, the effect of non-linear viscous roll damping should be taken into account. The damping coefficients that are derived within linear potential theory reflect the energy loss in the system due to generation of surface waves from the ship motions. However, in the case of roll motion some special treatment is necessary.

Viscous effects, such as the production of eddies around the hull, will mainly act as a damping mechanism, especially at large roll angles, and these effects should be included. Furthermore, the effects from roll damping devices, like bilge keels, should be evaluated. The roll damping shall be evaluated for the return period in question. The sea state in question may be considered when the damping is calculated.

L.5.3.4 Extreme wave induced responses

Extreme wave induced responses shall be long term responses. The standard method for calculation of the 100-year wave induced long term responses (e.g. vertical moment and shear force) shall use a scatter diagram in combination with a wave spectrum (e.g. Jonswap) and RAOs. Short term predictions of the responses are to be calculated for all significant wave heights (H_s) and peak (T_p) or zero up-crossing (T_z) periods combinations within the scatter diagram. A Weibull distribution is fitted to the resulting range of the

responses against probability of occurrence. This Weibull distribution is used to determine the response with a probability of occurrence corresponding to once every 100 years (100 year return period).

The short crested nature of the sea may be taken into account as a wave spreading function as given in NORSOK N-003.

The method described above is considered most accurate for estimating the 100-year value for the response in question. A short-term analysis based on the predicted 100-year wave height with corresponding T_z , will give comparable values if the following criteria are satisfied:

- The scatter diagram should be well developed with wave steepness approaching a constant value for the most extreme sea states
- The maximum wave induced response shall occur in a short term sea state which is retraceable from the scatter diagram with a 100 year wave height steepness
- The Weibull fit to predict the response shall be good fit with low residual sum (deviations from the regressed line)

It should be noted that the method of calculating the 100-year wave induced response for a short-term sea state based upon the predicted 100 year wave height with corresponding single value T_z or T_p does not recognise that there are a range of possible T_z (T_p). Therefore the range of possible T_z (T_p) within the 100-year return period should be investigated. The range of sea states with a 100 year return period should be found by developing a 100 year contour line from the scatter diagram. The wave bending moment should be calculated for several sea states on this contour line in order to find the maximum value.

L.5.3.5 Non-linear effects

Linear calculations as described above do not differentiate between sagging and hogging responses. The non-linearities shall be taken into account for the vertical bending moments and shear forces. The non-linear effects come from integration of the wave pressure over the instantaneous position of the hull relative to the waves, with the inclusion of bottom slamming, bow flare forces and deck wetness. These effects generally result in a reduction in hogging response and an increase in the sagging response (midship moment and shear force in the fore body) compared with the linear response.

L.6 ULTIMATE LIMIT STATES (ULS)

L.6.1 Global strength

L.6.1.1 General

Ultimate strength capacity check shall be performed for all structure contributing directly to the longitudinal and transverse strength of the ship. Structure to be checked is all plates and continuous stiffeners including the following structure:

- Main deck, bottom and inner bottom
- Ship side, inner ship side and longitudinal bulkheads
- Stringers and longitudinal girders
- Foundations of turret and topside structure
- Transverse bulkheads
- Transverse web frames

Global actions on the hull girder shall be calculated by direct wave analyses, see L.5.3. Longitudinal wave bending moments, shear forces and dynamic external sea pressures should be calculated by a wave load analysis. Design accelerations should also be based on a wave load analysis. For unconventional designs torsional effects may also be of importance. The design moments, forces, pressures and accelerations shall be calculated for a representative number of sections.

Stillwater shear forces and bending moments should be calculated by direct analyses, taking in account all relevant loading conditions.

Internal static and dynamic pressures can be calculated by simplified formulas, see L.6.1.4.

Local stresses should be calculated by FE-analyses of relevant parts of the ship, see L.5.2. All relevant load conditions should be taken in account.

The hull girder strength shall be evaluated for relevant combinations of still water bending moment and shear force, and wave induced bending moment and shear force. The wave-induced bending moments and shear forces shall be calculated by means of an analysis carried out utilising the appropriate statistical site specific environmental data. Relevant non-linear action effects shall be accounted for, see L.5.3.

The following design format shall be applied:

$$\gamma_S M_S + \gamma_W M_W < M_G / \gamma_M \quad (\text{L.6.1})$$

$$\gamma_S Q_S + \gamma_W Q_W < Q_G / \gamma_M \quad (\text{L.6.2})$$

where:

M_G	=	Characteristic bending moment resistance of the hull girder calculated as an elastic beam
M_S	=	Characteristic design still water bending moment based on actual cargo and ballast conditions
M_W	=	Characteristic wave bending moment. Annual probability of exceedance of 10^{-2} .
Q_G	=	Characteristic shear resistance of the hull girder calculated as an elastic beam
Q_S	=	Characteristic design still water shear force based on actual cargo and ballast conditions
Q_W	=	Characteristic wave shear force. Annual probability of exceedance of 10^{-2} .
γ_M	=	Material factor
γ_S	=	Permanent and variable action factor
γ_W	=	Environmental action factor

The action factors shall be in accordance with NORSOK N-001, see also Table L.6-1:

Table L.6-1

Action combinations	γ_s	γ_w
a	1.3	0.7
b	1.0	1.3

For combination of actions an action coefficient of 1.0 shall be applied for permanent actions where this gives the most unfavourable response.

The action coefficient for environmental actions may be reduced to 1.15 in action combination b, when the maximum stillwater bending moment represents between 20 and 50 % of the total bending moment. This reduction is applicable for the entire hull, both for shear forces and bending moments.

The action coefficient for permanent actions in action combination a, may be reduced to 1.2, if actions and responses are determined with great accuracy e.g. limited by the air-pipe height, and external static pressure to well defined draught.

Gross scantlings may be utilised in the calculation of hull structural resistance, provided a corrosion protection system in accordance with NORSOK N-001, is maintained.

The buckling resistance of the different plate panels shall be considered according to Section 8.

The global and local longitudinal stress components shall be combined in an appropriate manner with transverse stress and shear stress as relevant, see L.6.1.5.

L.6.1.2 Calculation of global stresses

Global longitudinal stresses should be calculated by FE-analysis, see L.5.2. For parallel parts of the midship, a simplified calculation of the hull girder section modulus can be performed. For units with moonpool / turret, a FE-analysis shall be carried out to describe the stress distribution in way of the openings, in particular in way of deck and bottom, and at termination of longitudinal strength elements. Global shear stresses shall be calculated considering the shear flow distribution in the hull.

L.6.1.3 Calculation of local transverse and longitudinal stresses

Local transverse and longitudinal stresses shall be superimposed with the global longitudinal stresses. Local stresses should be taken from a FEM analysis of a typical cargo area, see L.5.2. Phase information should be taken into account when available.

The FEM analysis shall include extreme hogging and sagging conditions as described in

Figure L.6-1. All relevant variations in tank filling should be considered in the analysis and reflected in the Operation Manual. The following stress components can be found from the FEM analysis, and should be combined with other stress components as described in L.6.1.5:

- Transverse stresses in webframes
- Double shell and double bottom stresses
- Local shear stresses in panels

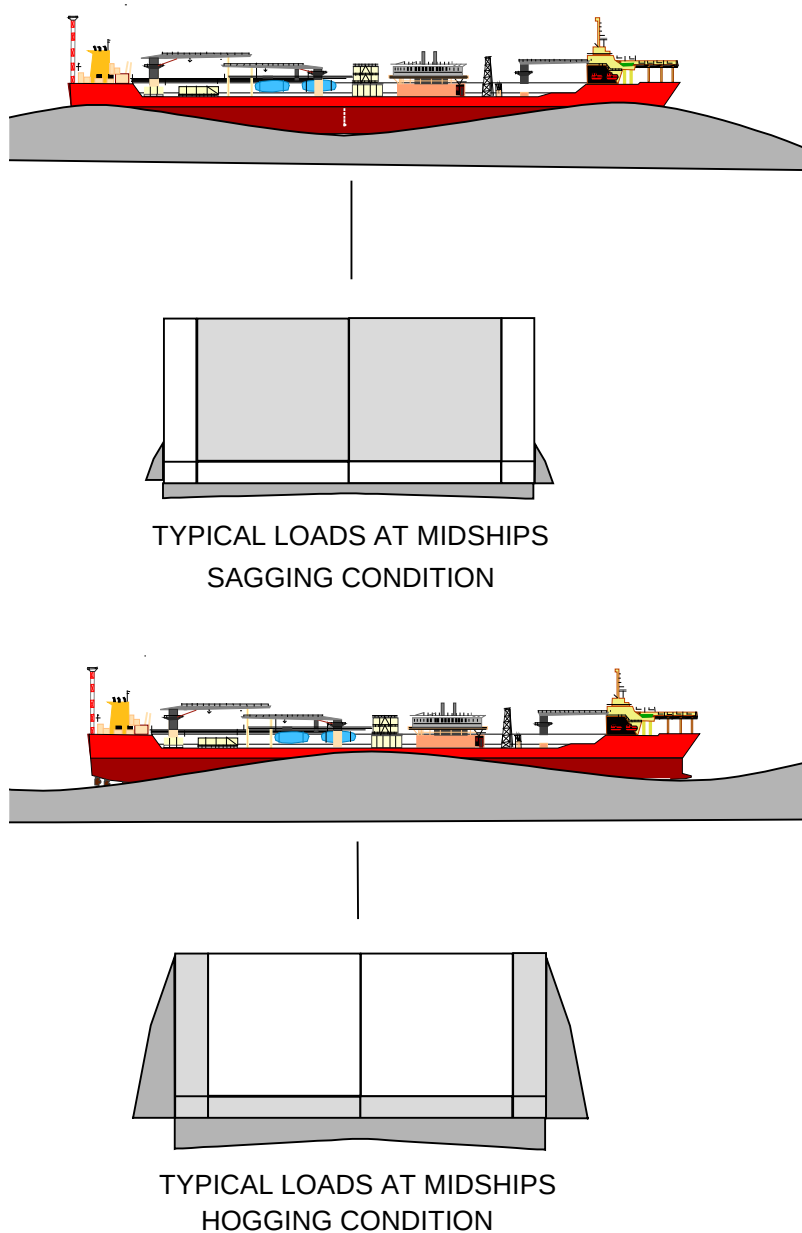


Figure L.6-1 Typical extreme sagging and hogging conditions

L.6.1.4 Calculation of local pressures

Sea pressure :

The dynamic sea pressure should be taken from a 3D hydrodynamic wave analysis. Action coefficients according to L.6.1.1 shall be considered.

Tank pressure:

The internal tank pressure shall be calculated in accordance with NORSOK N-003.

L.6.1.5 Combination of stresses

Total longitudinal design stress in the structure can be calculated as:

$$\sigma_{x, \text{Total}} = \sigma_{x, \text{Global}} + \sigma_{x, \text{Local}} \quad (\text{L.6.3})$$

Total transverse design stress in the structure is found directly from the FE-analysis:

$$\sigma_{y, \text{Total}} = \sigma_{y, \text{Local}} + \sigma_{y, \text{Global}} \quad (\text{L.6.4})$$

$\sigma_{y, \text{Global}}$ is normally relevant for the moonpool / turret area, and may be neglected in parallel parts of the midship.

Total design shear stress in the structure can be calculated as:

$$\tau_{x, \text{Total}} = \tau_{\text{Global}} + \tau_{\text{Local}} \quad (\text{L.6.5})$$

The combination of stresses should take into account actual stress directions and phase. However, if phase information is limited or uncertain, the maximum design value for each component may be combined as a 'worst-case' scenario.

Combination of typical stress components is shown in Figure L.6-2.

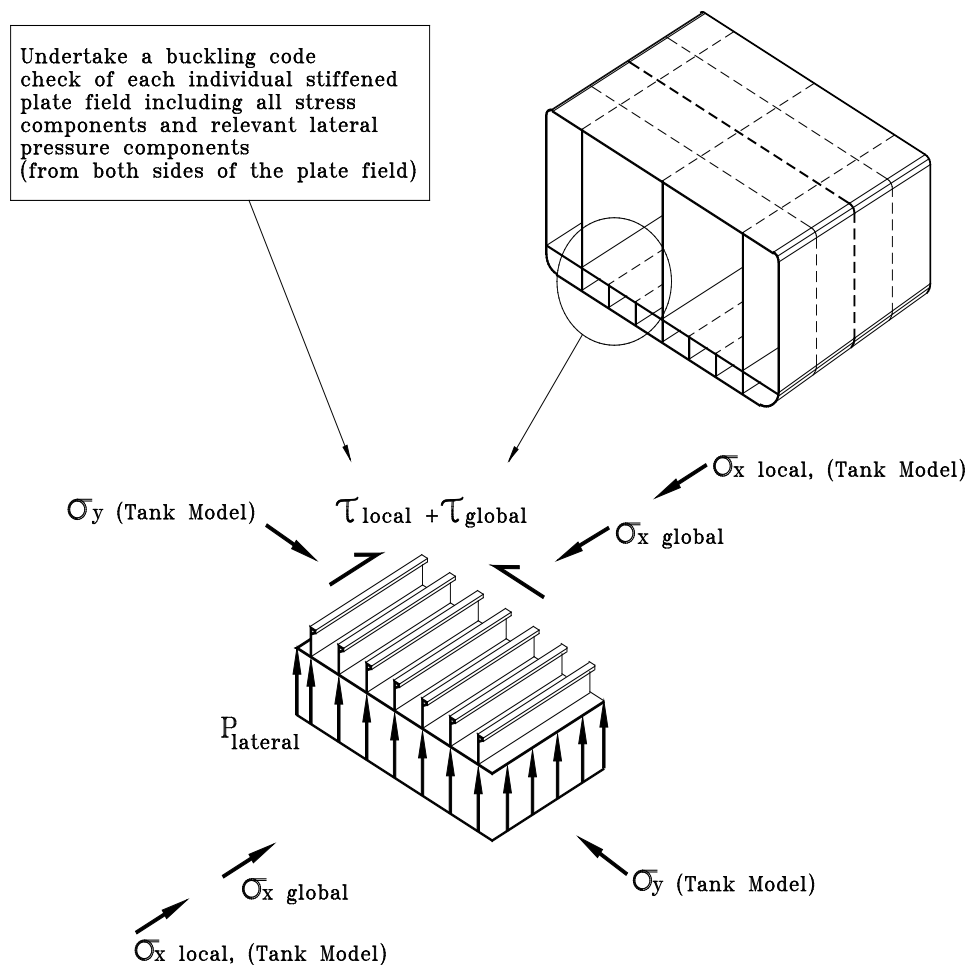


Figure L.6-2 Typical stress components acting on the hull beam.

L.6.1.6 Transverse structural strength

Transverse strength refers to the ability of the hull to resist lateral pressure and racking actions in combination with longitudinal action effects. This resistance is provided by means of transverse bulkheads, web frames, girders and stringers. Transverse strength should be evaluated using a finite element model of a specific portion of the hull, and the effects of process equipment deck actions should be included. The transverse strength shall meet the requirements given in L.6.1. The lateral pressure due to waves should be predicted using the hydrodynamic motion analysis approach, see L.5.3.

L.6.1.7 Capacity check

Capacity checks shall be performed in accordance with Chapter 6.

L.6.2 Local structural strength

Local structure not taking part of the overall strength of the unit may be designed in accordance with technical requirements given in the DNV Rules for Ships, Pt.3 Ch.1, /3/. Such structure may be in the following areas:

- Fore Ship.
- Aft Ship.
- Superstructure e.g. Process Deck and Accommodation.

For evaluation of slamming, sloshing and green sea effects, see L.4.5.2, L.4.5.3 and L.4.5.4.

L.6.3 Turret and turret area / moonpool

L.6.3.1 General

The following areas shall be considered as relevant, with respect to structural response from the mooring actions, combined with other relevant actions:

- Structure in way of moonpool opening in the ship hull.
- Turret structure including support, towards the unit hull.
- Structure in way of loading buoy support.
- Gantry structure including support.

L.6.3.2 Structure in way of moonpool opening in the unit hull

The structural strength shall be evaluated considering all relevant, realistic action conditions and combinations, see L.4. In particular action combinations due to the following shall be accounted for in the design:

- Turret bearing reactions.
- Overall hull bending moments and shear forces.
- Internal and external pressure actions, covering the intended range of draughts and action conditions, including non-symmetric cases as applicable.

Finite element analyses shall be carried out. Particular attention shall be given to critical interfaces. Functional limitations (bearing function) and structural strength shall be evaluated.

Continuity of primary longitudinal structural elements should be maintained as far as practicable in way of the turret opening. Reductions in hull section modulus shall be kept at a minimum and compensation shall be made where necessary.

L.6.3.3 Turret structure

A FEM analysis of the turret structure shall be performed, see L.5.2.4, demonstrating that the structural strength of the turret is acceptable. The structural strength shall be evaluated considering all relevant, realistic action conditions and combinations, see L.4. In particular, action combinations due to the following actions (as applicable) shall be accounted for in the design:

- Unit motion induced accelerations
- Mooring actions
- Riser actions
- Turret bearing reactions (calculated based on all the relevant actions on the turret)
- Moonpool deformations (based on hull bending moments and shear forces)
- Internal and external pressure actions, covering the intended range of draughts and load conditions including non-symmetric cases as applicable. Filling of void spaces to be accounted for (if relevant).
- Local actions from equipment and piping system (weight, thermal expansion, mechanical actions)
- Green seas
- Wave slamming

Boundary conditions for the model shall reflect the true configuration of the interface towards the unit.

Local analyses, (model level 3 & 4, see L.5.2.5 and L.5.2.6) shall be performed for structural areas, which are critical for the structural integrity of the turret. The following list contains typical areas which should be considered:

- Structure in vicinity of riser connection(s)
- Riser hang-off structure
- Structure in way of fairleads
- Hang-off structure for anchor line
- Local structure transferring bearing reactions
- Chain lockers
- Pipe supports (single supports and complex structures)
- Equipment supports
- Foundation for transfer system (especially for swivel solutions)
- Lifting appliances and pad-eyes including structure in way of these

L.6.4 Topside facilities structural support

The structural strength of topside facilities structural support shall be evaluated considering all relevant action conditions and combinations of actions. Scantlings shall be determined on the basis of criteria, which combine, in a rational manner, the effects of global and local responses for each structural element.

The following actions shall be considered:

- Permanent actions (weight of structures, process and drilling equipment, piping etc.)
- Live actions (equipment functional actions related to liquid, machinery, piping reaction forces, helicopter, cranes etc.)
- Wave actions
- Wave accelerations (inertia actions)
- Hull deflections due to tank filling, wave, temperature differences etc.
- Wind
- Snow and ice
- Green sea

L.6.5 Transit conditions

For self-propelled units, requirements given by the RCS (and the flag state) for seagoing ships cover the transit condition. Due consideration should be made with respect to dynamic actions, in particular due to roll motion, on the topside equipment.

L.7 FATIGUE LIMIT STATES (FLS)

L.7.1 General

General requirements and guidance concerning fatigue criteria are given in Chapter 8, and Annex C.

Evaluation of the fatigue limit states shall include consideration of all significant actions contributing to fatigue damage both in non-operational and operational design conditions.

The minimum fatigue life of the unit (before the design fatigue factor is considered) should be based upon a period of time not being less than the planned life of the structure.

Local effects, for example due to :

- slamming,
- sloshing,
- vortex shedding,
- dynamic pressures, and,
- mooring and riser systems,

shall be included in the fatigue damage assessment when relevant.

Calculations carried out in connection with the fatigue limit state may be undertaken without deducting the corrosion additions, provided a corrosion protection system in accordance with NORSOK, N-001 is maintained.

In the assessment of fatigue resistance, relevant consideration shall be given to the effects of stress raisers (concentrations) including those occurring as a result of :

- fabrication tolerances, (including due regard to tolerances in way of connections of large structural sections),
- cut-outs,
- details at connections of structural sections (e.g. cut-outs to facilitate construction welding).

L.7.2 Design fatigue factors

Criteria related to Design Fatigue Factors, are given in the Chapter 8. When determining the appropriate Design Fatigue Factor for a specific fatigue sensitive location, consideration shall be given to the following :

- Consideration of economic consequence of failure may indicate the use of larger design factors than those provided for as minimum factors.
- The categorisation : 'Accessible / Above splash zone' is, intended to refer to fatigue sensitive locations where the possibility for close-up, detailed inspection in a dry and clean condition exists. When all of these requirements are not fulfilled, the relevant design fatigue factor should be considered as being that appropriate for 'Accessible / Below splash zone', or, 'No access or in the splash zone' as relevant to the location being considered.
- Evaluation of likely crack propagation paths (including direction and growth rate related to the inspection interval), may indicate the use of a different Design Fatigue Factor than that which would be selected when the detail is considered in isolation, such that :
 - Where the likely crack propagation indicates that a fatigue failure, from a location satisfying the requirements for a 'Non-substantial' consequence of failure, may result in a 'Substantial' consequence of failure, such fatigue sensitive location is itself to be deemed to have a 'Substantial' consequence of failure.
 - Where the likely crack propagation is from a location satisfying the requirement for a given 'Access for inspection and repair' category to a structural element having another access categorisation, such location is itself to be deemed to have the same categorisation as the most demanding category when considering the most likely crack path. For example, a weld detail on the inside (dry space) of a submerged shell plate should be allocated the same Fatigue Design Factor as that relevant for a similar weld located externally on the plate –see Figure L.7-1.

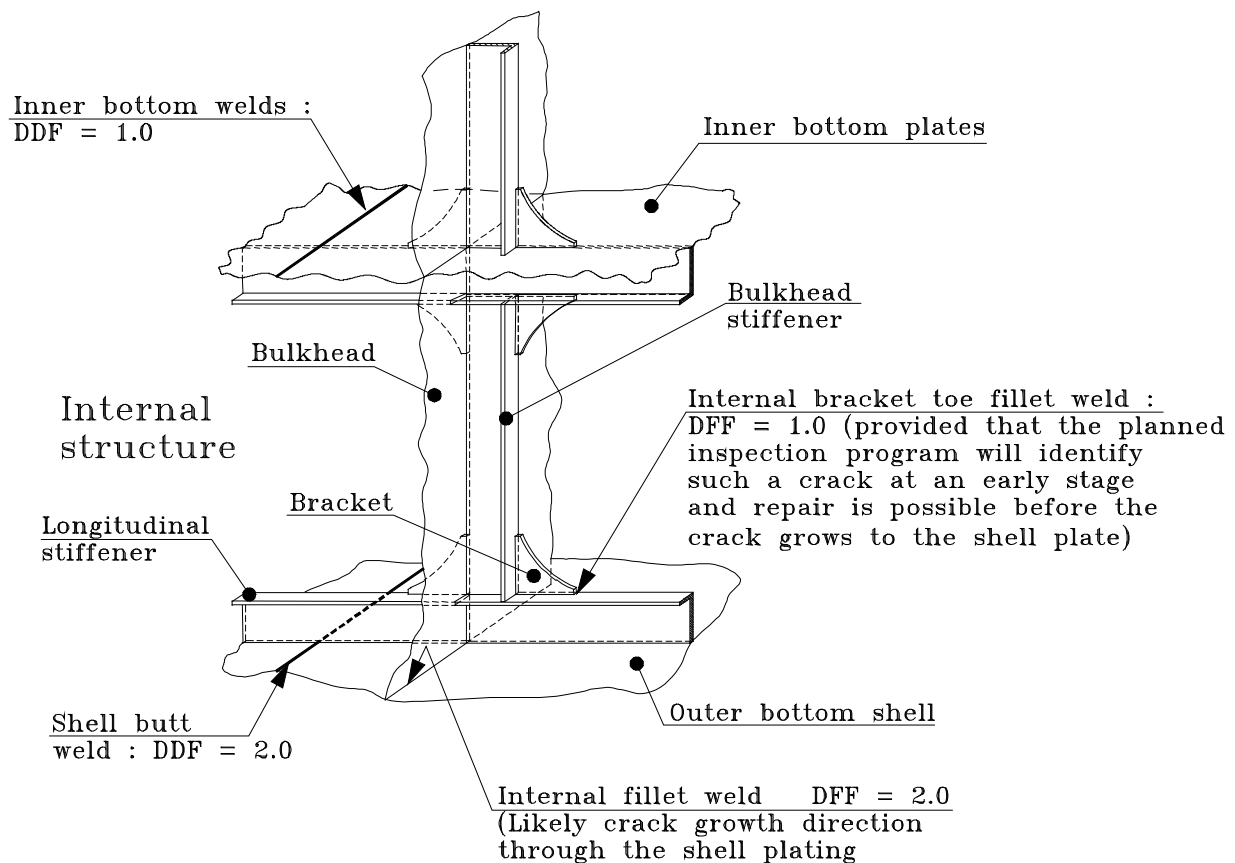


Figure L.7-1 Example illustrating considerations relevant for selection of design fatigue factors (DDF's) in locations considered to have 'Non-substantial' consequence of failure.

Notes:

1. Due to economic considerations (e.g. cost of repair to an external underwater structural element) the DDF's assigned a value of 2 in Figure L.7-1 may be considered as being more appropriately assigned a value of 3.)
2. The unit may be considered as "accessible and above the splash zone" (DDF = 1.0), see Annex C, if the survey extent e.g. given for main class (see DNV Ship Rules Pt.7 Ch.2) is followed i.e. drydocking for inspection and maintenance every 5 years.

L.7.3 Splash zone

The definition of 'splash zone' as given in NORSOK N-003, relates to a highest and lowest tidal reference. For ship shaped units, for the evaluation of the fatigue limit state, reference to the tidal datum should be substituted by reference to the draught that is intended to be utilised when condition monitoring is to be undertaken. The requirement that the extent of the splash zone is to extend 5 m above and 4 m below this draught may then be applied. (For application of requirements to corrosion addition, however, the normal operating draught should generally be considered as the reference datum).

If significant adjustment in draught of the unit is possible in order to provide for satisfactory accessibility in respect to inspection, maintenance and repair, account may be taken of this possibility in the determination of the Design Fatigue Factors. In such cases however, a margin of minimum 1 meter in respect to the minimum inspection draught should be considered when deciding upon the appropriate Design Fatigue Factor in relation to the criteria for 'Below splash zone' as opposed to 'Above splash zone'. Where draft adjustment possibilities exist, a reduced extent of splash zone may be applicable. Consideration should be given to operational requirements that may limit the possibility for ballasting / deballasting operations.

When considering utilisation of Remotely Operated Vehicle (ROV) inspection consideration should be given to the limitations imposed on such inspection by the action of water particle motion (e.g. waves). The practicality of such a consideration may be that effective underwater inspection by ROV, in normal sea conditions, may not be achievable unless the inspection depth is at least 10 metres below the sea surface.

L.7.4 Structural details and stress concentration factors

Fatigue sensitive details in the FPSO should be documented to have sufficient fatigue strength. Particular attention should be given to connection details of the following:

- Integration of the mooring system with hull structure
- Main hull bottom, side and decks
- Main hull longitudinal stiffener connections to transverse frames and bulkheads
- Main hull attachments; seats, supports etc.
- Openings in main hull
- Transverse frames
- Flare tower
- Riser interfaces
- Major process equipment seats

Selections of local details and calculations of stress concentration factors may be undertaken in accordance with DNV Classification Note 30.7, /11/. For details not covered in this document, stress concentration factors should be otherwise documented. Detailed finite element analysis may be utilized for determination of SCF's, according to procedure given in DNV Classification Note 30.7, /11/.

L.7.5 Design actions and calculation of stress ranges

L.7.5.1 Fatigue actions

An overview of fatigue actions is given in L.4.7. Site specific environmental data shall be used for calculation of long term stress range distribution. For units intended for multi field developments the site specific environmental actions for each field should be utilised considering the expected duration for each field. The most onerous environmental actions may be applied for the complete lifetime of the unit, as a conservative approach.

A representative range of action conditions shall be considered. It is generally acceptable to consider two action conditions, typically: ballast condition and the fully loaded condition, with appropriate amount of time at each condition, normally 50 % for each condition unless otherwise documented.

An appropriate range of wave directions and wave energy spreading shall be considered. For weather waning units, and in absence of more detailed documentation, the head sea direction shall be considered with the spreading taken as the most unfavourable between $\cos^2 - \cos^{10}$, see NORSOK, N-003. Maximum spacing should be 30 degrees. Smaller spacing should be considered around the head-on heading, e.g. 15 degrees.

Typically, most unfavourable spreading will be $\cos^2$ for responses dominated by beam sea e.g. external side pressure, and $\cos^{10}$ for responses dominated by head sea e.g. global wave bending moment.

The following dynamic actions shall be included in a FLS analysis as relevant:

- Global wave bending moments
- External dynamic pressure due to wave and ship motion
- Internal dynamic pressure due to ship motion
- Sloshing pressures due to fluid motion in tanks for ships with long or wide tanks
- Loads from equipment and topside due to ship motion and acceleration

L.7.5.2 Topside structures

The following actions shall be considered for the topside structure:

- Hull deformations due to wave bending moment acting on the hull
- Wave induced accelerations (inertia actions)
- Vortex induced vibrations from wind
- Vibrations caused by operation of topside equipment

Additionally, the following low cycle actions should be considered where relevant for the topside structure:

- Hull deformations due to temperature differences
- Hull deformations due to change in filling condition e.g. ballasting / deballasting

The relevance of combining action components is dependent on the structural arrangement of the topside structure. Relevant stress components, both high cycle and low cycle, shall be combined, including phase

information, when available. If limited phase information is available, the design may be based on 'worst-case' action conditions, by combination of maximum stress for each component.

L.7.5.3 Turret structure

The turret structure will normally be exposed to high dynamic action level. The choice of fatigue design factor for the turret should reflect the level of criticality and the access for inspection at the different locations, L.7.2.

The following actions shall be considered for the fatigue design of turret structures:

- Dynamic fluctuations of mooring line tension.
- Dynamic actions (tension and bending moment) from risers
- Varying hydrodynamic pressure due to wave action
- Varying hydrodynamic pressure due to unit accelerations (including added mass effects)
- Reactions in the bearing structure due to the other effects
- Inertia actions due to unit accelerations
- Fluctuating reactions in pipe supports due to thermal and pressure induced pipe deflections

Typical critical areas for fatigue evaluation listed in L.6.3.3.

L.7.5.4 Calculation of global dynamic stress ranges

Global stress ranges shall be determined from the global hull bending moments. If applicable, both vertical and horizontal bending moments shall be included. Shear lag effects and stress concentrations shall be considered.

L.7.5.5 Calculation of local dynamic stress ranges

Local stress ranges are determined from dynamic pressures acting on panels, accelerations acting on equipment and topside and other environmental actions resulting in local stresses to part of the structure.

Dynamic pressures shall be calculated from a 3D sink-source wave load analysis. The transfer function for the dynamic pressure could either be used directly to calculate local stress transfer functions and combined with the global stress transfer function, or a long-term pressure distribution could be calculated. As a minimum, the following dynamic pressures components shall be considered:

- Double hull stresses due to bending of double hull sections between bulkheads
- Panel stresses due to bending of stiffened plate panels
- Plate bending stresses due to local plate bending

For a description of calculation of local stress components, reference is made to DNV Classification Note 30.7, /11/.

L.7.5.6 Combination of stress components

Global and local stresses should be combined to give the total stress range for the detail in question. In general, the global and the local stress components differ in amplitude, phase and location. The method of combining these stresses for the fatigue damage calculation will depend on the location of the structural detail. A method for combination of actions is given in DNV Classification Note 30.7, /11/.

L.7.6 Calculation of fatigue damage

L.7.6.1 General

The basis for determining the acceptability of fatigue resistance, with respect to wave actions, shall be appropriate stochastic fatigue analyses. The analyses shall be undertaken utilising relevant site specific environmental data and take appropriate consideration of both global and local (e.g. pressure fluctuation) dynamic responses. (These responses do not necessarily have to be evaluated in the same model but the cumulative damage from all relevant effects should be considered when evaluating the total fatigue damage.)

Simplified fatigue analyses may form the basis of a 'screening' process to identify locations for which a detailed, stochastic fatigue analysis should be undertaken. Such simplified fatigue analysis shall be calibrated, see L.7.6.2.

Local, detailed FE-analysis (e.g. unconventional details with insufficient knowledge about typical stress distribution) should be undertaken in order to identify local stress distributions, appropriate SCF's, and/or

extrapolated stresses to be utilised in the fatigue evaluation, see Annex C for further details. Dynamic stress variations through the plate thickness shall be documented and considered in such evaluations.

Explicit account shall be taken of any local structural details that invalidate the general criteria utilised in the assessment of the fatigue strength. Such local details may, for example be access openings, cut-outs, penetrations etc. in structural elements.

Principal stresses (see Annex C) should be utilised in the evaluation of fatigue responses.

L.7.6.2 Simplified fatigue analysis

Provided that the provisions stated in L.7.6.1, are satisfied, simplified fatigue analysis may be undertaken in order to establish the general acceptability of fatigue resistance. In all cases when a simplified fatigue analysis is utilized a control of the results of the simplified fatigue evaluation, compared to the stochastic results, shall be documented to ensure that the simplified analysis provides for a conservative assessment for all parts of the structure being considered.

The fatigue damage may be calculated based on a cumulative damage utilizing Miner-Palmgren summation, applying a Weibull distribution to describe the long-term stress range. The stress range shape parameter and the average zero-crossing frequency may be taken from the long-term stress distribution, utilizing the stress transfer function and environmental data for the operating area. A simplified method of fatigue analysis is described in DNV Classification Note 30.7, /11/.

L.7.6.3 Stochastic fatigue analysis

Stochastic fatigue analyses shall be based upon recognised procedures and principles utilising relevant site specific data.

Providing that it can be satisfactorily documented, scatter diagram data may be considered as being directionally specific. In such cases, the analyses shall include consideration of the directional probability of the environmental data. Relevant wave spectra shall be utilised. Wave energy spreading may be taken into account if relevant.

Structural response shall be determined based upon analyses of an adequate number of wave directions. Generally a maximum spacing of 30 degrees should be considered. Transfer functions should be established based upon consideration of a sufficient number of periods, such that the number, and values of the periods analysed :

- adequately cover the site specific wave data,
- satisfactorily describe transfer functions at, and around, the wave 'cancellation' and 'amplifying' periods. (Consideration should be given to take account that such 'cancellation' and 'amplifying' periods may be different for different elements within the structure), and,
- satisfactorily describe transfer functions at, and around, the relevant excitation periods of the structure.

The method is described in DNV Classification Note 30.7, /11/.

L.8 ACCIDENTAL LIMIT STATES (ALS)

Minimum impact energy levels, action combinations and allowable utilisation levels shall comply with the requirements given in NORSOK, N-003. Relevant actions shall be determined based on a Risk Analysis.

L.8.1 Dropped objects

The possibility of dropped objects impacting on the structural components of the unit shall be considered in design. Resistance to dropped objects may be accounted for by indirect means, such as, using redundant framing configurations, and, materials with sufficient toughness in affected areas.

The masses of the dropped objects from crane operation to be considered for design of units are normally taken based upon operational hook actions of the platform crane. Critical areas of dropped objects are to be determined based on crane operation sectors, crane reach and actual movement of actions assuming a drop direction within an angle to the vertical direction of 5° in air and 15° in water.

L.8.2 Fire

General guidance and requirements concerning accidental limit state events involving fire are given in Chapter 9 and Annex A.

L.8.3 Explosion

In respect to design considering actions resulting from explosions one, or a combination of the following main design philosophies are relevant ;

- Ensure that the probability of explosion is reduced to a level where it is not required to be considered as a relevant design action.
- Ensure that hazardous locations are located in unconfined (open) locations and that sufficient shielding mechanisms (e.g. blast walls) are installed.
- Locate hazardous areas in partially confined locations and design utilising the resulting relatively small overpressures.
- Locate hazardous areas in enclosed locations and install pressure relief mechanisms (e.g. blast panels) and design for the resulting overpressure.

Structural design accounting for large plate field rupture resulting from explosion actions should normally be avoided due to the uncertainties of the actions and the consequences of the rupture itself.

Structural support of the blast walls and the transmission of the blast action into main structural members shall be evaluated. Effectiveness of connections and the possible outcome from blast, such as flying debris, shall be considered.

L.8.4 Collision

Resistance to unit collisions may be accounted for by indirect means, such as, using redundant framing configurations, and, materials with sufficient toughness in affected areas.

Collision impact shall be considered for all elements of the unit, which may be impacted by sideways, bow or stern collision. The vertical extent of the collision zone shall be based on the depth and draught of attending units and the relative motion between the attending units and the unit.

To avoid possible penetration of a cargo tank, the side structure of the unit shall be capable of absorbing the energy of a vessel collision with an annual probability of 10^{-4} .

L.8.5 Unintended flooding

A procedure describing actions to be taken after relevant unintended flooding shall be prepared. Structural aspects related to counterflooding (if relevant) shall be investigated.

The unintended flooding conditions shall be considered in the turret and topside structure design.

L.8.6 Loss of heading control

For units normally operated with heading control, either by weather vaning or by thruster assistance, the effect of loss of the heading control shall be evaluated.

The loss of heading control condition shall be considered in the topside and turret structure design.

L.9 COMPARTMENTATION & STABILITY

L.9.1 General

In the assessment of compartmentation and stability of a ship shaped unit, consideration shall be given to all relevant detrimental effects, including those resulting from:

- environmental actions,
- relevant damage scenarios,
- rigid body motions,
- the effects of free-surface, and,
- boundary interactions (e.g. mooring and riser systems).

An inclining test should be conducted when construction is as near to completion as practical in order to accurately determine the unit's mass and position of the centre of gravity. Changes in mass conditions after the inclining test should be carefully accounted for. The unit's mass and position of the centre of gravity shall be reflected in the unit's Operating Manual.

L.9.2 Compartmentation and watertight integrity

General requirements relating to compartmentation of ship shaped units are given in ISO 19900, /6/. Detailed provisions given in the IMO MODU Code relating to subdivision and freeboard should be complied with. Special provisions relating to the Norwegian petroleum activities are given in NORSOK N-001. Additionally, the design criteria for watertight and weathertight integrity as stated in the relevant rules of Recognised Classification Societies should normally be satisfied.

The number of openings in watertight structural elements shall be kept to a minimum. Where penetrations are necessary for access, piping, venting, cables etc., arrangements shall be made to ensure that the watertight integrity of the structure is maintained.

Where individual lines, ducts or piping systems serve more than one watertight compartment, or are within the extent of damage resulting from a relevant accidental event, arrangements shall be provided to ensure that progressive flooding will not occur.

L.9.3 Stability

Stability of a ship shaped unit shall satisfy the requirements as stated in the IMO MODU Code. Special provisions relating to the Norwegian petroleum activities are given in NORSOK N-001.

Adequacy of stability shall be established for all relevant in-service and temporary phase conditions. The assessment of stability shall include consideration of both the intact and damaged conditions.

L.10 SPECIAL CONSIDERATION

L.10.1 Structural details

For the design of structural details consideration should be given to the following:

- The thickness of internal structure in locations susceptible to excessive corrosion.
- The design of structural details, such as those noted below, against the detrimental effects of stress concentrations and notches:
 - Details of the ends and intersections of members and associated brackets.
 - Shape and location of air, drainage, and lightening holes.
 - Shape and reinforcement of slots and cut-outs for internals.
 - Elimination or closing of weld scallops in way of butts, "softening" bracket toes, reducing abrupt changes of section or structural discontinuities.
- Proportions and thickness of structural members to reduce fatigue damage due to engine, propeller or wave-induced cyclic stresses, particularly for higher strength steel members.

L.10.2 Positioning of superstructure

Living quarters, lifeboats and other means of evacuation shall be located in non-hazardous areas and be protected and separated from production, storage, riser and flare areas.

Superstructure shall be located such that a satisfactory level of separation and protection is provided for.

L.10.3 Structure in way of a fixed mooring system

Local structure in way of fairleads, winches, etc. forming part of the position mooring system is, as a minimum, to be capable of withstanding forces equivalent to 1.25 times the breaking strength of the individual mooring line. The strength evaluation should be undertaken utilising the most unfavourable operational direction of the anchor line. In the evaluation of the most unfavourable direction, account shall be taken of relative angular motion of the unit in addition to possible line lead directions.

L.10.4 Inspection and maintenance

The structural inspection and maintenance programmes shall be consistent with NORSOK N-001.

L.10.5 Facilities for inspection on location

Inspections may be carried out on location based on approved procedures outlined in a maintenance system and inspection arrangement, without interrupting the function of the unit. The following matters should be taken into consideration to be able to carry out condition monitoring on location:

- Arrangement for underwater inspection of hull, propellers, thrusters, rudder and openings affecting the units seaworthiness.
- Means of blanking of all openings including side thrusters.
- Use of corrosion resistant materials for shafts, and glands for propeller and rudder.
- Accessibility of all tanks and spaces for inspection.
- Corrosion protection of hull.
- Maintenance and inspection of thrusters.
- Ability to gas free and ventilate tanks
- Provisions to ensure that all tank inlets are secured during inspection
- Measurement of wear in the propulsion shaft and rudder bearings.
- Testing facilities of all important machinery.

L.10.6 Action monitoring

An on-board loading instrument complying with the requirements of the DNV Ship Rules, /3/ shall be installed in order to be able to monitor still water bending moments and shear forces, and the stability of the unit.

The limitations for still water bending moments and shear forces shall be in accordance with maximum allowable stillwater bending moments and shear forces specified in the Operating Manual.

L.10.7 Corrosion protection

The corrosion protection arrangement shall be consistent with the references listed in NORSOK N-001.

L.11 DOCUMENTATION

L.11.1 General

Adequate planning shall be undertaken in the initial stages of the design process in order to obtain a workable and economic structural solution to perform the desired function. As an integral part of this planning, documentation shall be produced identifying design criteria and describing procedures to be adopted in the structural design of the unit.

Applicable codes, standards and regulations should be identified at the commencement of the design.

When the design has been finalised, a summary document containing all relevant data from the design and fabrication phase shall be produced.

Design documentation (see below) shall, as far as practicable, be concise, non-voluminous, and, should include all relevant information for all relevant phases of the lifetime of the unit.

General requirements to documentation relevant for structural design are given in NORSOK, N-001, Section 5.

L.11.2 Design basis

A Design Basis Document shall be created in the initial stages of the design process to document the basis criteria to be applied in the structural design of the unit.

A summary of those items normally to be included in the Design Basis document is included below.

Unit description and main dimensions

A summary description of the unit, including :

- general description of the unit (including main dimensions and draughts),
- main drawings including :
 - main structure drawings (This information may not be available in the initial stage of the design),
 - general arrangement drawings,
 - plan of structural categorisation,
 - load plan(s), showing description of deck uniform (laydown) actions,
 - capacity (tank) plan,
- service life of unit,
- position keeping system description.

Rules, regulations and codes

A list of all relevant, applicable, Rules, Regulations and codes (including revisions and dates), to be utilised in the design process.

Environmental design criteria

Environmental design criteria (including all relevant parameters) for all relevant conditions, including :

- wind, wave, current, snow and ice description for 10^{-1} , 10^{-2} and 10^{-4} annual probability events,
- design temperatures.

Stability and compartmentation

Stability and compartmentation design criteria for all relevant conditions including :

- external and internal watertight integrity plan,
- lightweight breakdown report,
- design loadcase(s) including global mass distribution,
- damage condition waterlines. (This information may not be available in the initial stage of the design.)

Temporary phases

Design criteria for all relevant temporary phase conditions including, as relevant :

- limiting permanent, variable, environmental and deformation action criteria,
- procedures associated with construction, (including major lifting operations),

- relevant ALS criteria.

Operational design criteria

Design criteria for all relevant operational phase conditions including :

- limiting permanent, variable, environmental and deformation action criteria,
- designing accidental event criteria (e.g. collision criteria),
- tank loading criteria (all tanks) including a description of system, with :
 - loading arrangements
 - height of air pipes,
 - loading dynamics,
 - densities,
- mooring actions.

In-service inspection criteria

A description of the in-service inspection criteria and general philosophy (as relevant for evaluating fatigue allowable cumulative damage ratios).

Miscellaneous

A general description of other essential design information, including :

- description of corrosion allowances, where applicable.

L.11.3 Design brief

A Design Brief Document shall be created in the initial stages of the design process. The purpose of the Design Brief shall be to document the intended procedures to be adopted in the structural design of the unit. All applicable limit states for all relevant temporary and operational design conditions shall be considered in the Design Brief.

A summary of those items normally to be included in the Design Brief Document is included below.

Analysis models

A general description of models to be utilised, including description of :

- global analysis model(s),
- local analysis model(s),
- loadcases to be analysed.

Analysis procedures

A general description of analytical procedures to be utilised including description of procedures to be adopted in respect to :

- the evaluation of temporary conditions,
- the consideration of accidental events,
- the evaluation of fatigue actions,
- the establishment of dynamic responses (including methodology, factors, and relevant parameters),
- the inclusion of 'built-in' stresses (if relevant),
- the consideration of local responses (e.g. those resulting from mooring and riser actions, ballast distribution in tanks as given in the operating manual etc.)

Structural evaluation

A general description of the evaluation process, including :

- description of procedures to be utilised for considering global and local responses,
- description of fatigue evaluation procedures (including use of design fatigue factors, SN-curves, basis for stress concentration factors (SCF's), etc.).
- description of procedures to be utilised for code checking.

L.11.4 Documentation for operation (DFO)

Documentation requirements covering structural design, as applicable in the operational phase of the unit (e.g. the DFI resume), are given in NORSOK Z-001.

L.12 REFERENCES

- /1/ 'Offshore Installations: Guidance on design, construction and certification', U.K., Health & Safety Executive, Fourth Edition, HMSO
- /2/ DNV-OS-C101 Offshore Standard. Design of Offshore Steel Structures, General. DNV October 2000.
- /3/ Rules for Classification of Ship, latest valid edition, DNV
- /4/ Regulations relating to design and outfitting of facilities etc. in the petroleum activities. Issued by the Norwegian Petroleum Directorate.
- /5/ Regulations of 20 Dec. 1991, No.878, concerning stability, watertight subdivision and watertight / weathertight closing means on mobile offshore units, Norwegian Maritime Directorate.
- /6/ ISO 19900 Petroleum and Natural Gas Industries – Offshore Structures, General Requirements.
- /7/ Salvesen, N., Tuck, E. O. and Faltinsen, O., "Ship Motions and Sea Loads".Trans. SNAME, Vol 78, 1970.
- /8/ Tanaka, N., "A Study of the Bilge Keels, Part 4, on the Eddy-Making Resistance to the Rolling of a Ship Hull". Japan Society of Naval Architects, Vol. 109, 1960.
- /9/ Torsethaugen, Knut, "Model for a Doubly Peaked Spectrum". Sintef Report No. SFT22 96204 dated 1996-02-20.
- /10/ NORSOK Z-001 Documentation for Operation (DFO)
- /11/ DNV Classification Note 30.7: Fatigue Assessment of Ship Structures
- /12/ DNV-OS-C102 Offshore Standard. Structural Design of Offshore Ships. DNV October 2011.

L.13 COMMENTARY

Comm. L.4.5.2 Sloshing actions in tanks

The non-impact sloshing loads in the DNV Rules are given at annual probability level 10^{-4} and represent the sloshing loads generally applicable for a FLS structural evaluation. These loads are governed by the inertia forces induced by the liquid in the tank. This means that the Weibull slope parameter applicable for a FLS structural evaluation, will be approximately equal to the Weibull slope parameter of the liquid acceleration in the longitudinal direction for the longitudinal sloshing mode and in the transverse direction for the transverse sloshing mode. Based on analysis of several offshore vessels ranging in length from 100-260m, a Weibull slope parameter $h=1.0$ has been determined as being appropriate for an FLS structural evaluation of ship shaped offshore units, both in the longitudinal sloshing mode and in the transverse sloshing mode. The non-impact sloshing loads should be applied on strength members as indicated in DNV Ship Rules Pt.3, Ch.1, Sec.4 C300 Liquid in tanks.

The sloshing impact loads in the DNV Rules are given at annual probability level 10^{-8} (20 year return period) and represent the sloshing impact loads generally applicable for an ULS structural evaluation. In the period 1990-1994, DNV performed a significant amount of sloshing model tests, using irregular excitation, with the main focus on sloshing impact loads. The duration of the tests was typically 2-4 hours in model scale. Based on statistical analysis of these model tests, a Weibull slope parameter $h \approx 1.0$ has been determined as being representative for sloshing impact loading. To arrive at 100 year return period impact pressures, the 10^{-8} values should be multiplied by a factor 1.15. The impact sloshing loads should be applied on strength members as indicated in DNV Ship Rules Pt.3, Ch.1, Sec.4 C300 Liquid in tanks.

A wave which will impose the maximum wave bending moment on the vessel will have a wavelength of the order ships length. Provided that separation between the period of encounter for this wave T_e and the natural period T of the fluid in the tank is sufficient ($T \leq 0.75T_e$), use of a reduced wave bending moment in calculating the allowable stress used in connection with sloshing loads for deck longitudinals is acceptable. The strength calculation for deck longitudinals may then be based on a wave bending moment reduced by 25% relative to the design wave bending moment.

The applicable number of cycles in a fatigue evaluation will in principle depend on the vessel pitch response period for the longitudinal sloshing mode and on the vessel roll response period for the transverse sloshing mode. A simplified expression for the applicable response period, both for the longitudinal and transverse sloshing mode, may be found in DNV Class Note 30.7, Section 2.1.2, $T_{\text{resp}} = 4 \cdot \log_{10}(L)$, where L is the rule ship length.

Comm. L.4.5.3 Green water effect

It is recommended that provisions are made during model testing for suitable measurements to determine design pressures for local structural design. This implies that model tests should be performed at design draught, for sea states with a spectrum peak period approximately 70-100% of the pitch resonance period of the vessel. The vessel model should be equipped with load cells on the weather deck at positions of critical structural members or critical topside equipment.

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DESIGN OF STEEL STRUCTURES
ANNEX M
SPECIAL DESIGN PROVISIONS FOR COLUMN STABILIZED UNITS

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M.1 Introduction

M.1.1 General

This annex provides requirements and guidance to the structural design of column stabilised units, constructed in steel, in accordance with the provisions of this NORSOK standard.

The requirements and guidance documented in this annex are generally applicable to all configurations of column stabilised units, including those with

- ring (continuous) pontoons,
- twin pontoons or,
- multi-footing arrangements.

Such units may be kept on station by either a passive mooring system (e.g. anchor lines), or an active mooring system (e.g. thrusters), or a combination of these methods.

Requirements concerning mooring and riser systems are not considered in this annex.

A column stabilised unit may be designed to function in a number of modes, e.g. transit, operational and survival. Limiting design criteria for going from one mode of operation to another mode of operation shall be clearly established and documented. Such limiting design criteria shall include relevant consideration of the following items:

- intact condition, structural strength;
- damaged condition, structural strength;
- air gap;
- compartmentation and stability.

For novel designs, or unproved applications of designs where limited or no direct experience exists, relevant analyses and model testing, shall be performed which clearly demonstrate that an acceptable level of safety is obtained.

M.1.2 Definitions

Column stabilised unit: A floating unit that can be relocated. A column stabilised unit normally consists of a deck structure with a number of widely spaced, large diameter, supporting columns that are attached to submerged pontoons.

Notes:

1. In the context of this annex the term "column stabilised unit" is often abbreviated to the term "unit".
2. The definition for a column stabilised unit is in accordance with ISO 19900 definition for a "semi-submersible" unit.

M.1.3 Non-operational phases

The structure shall be designed to resist relevant actions associated with conditions that may occur during all stages of the life-cycle of the unit. Such stages may include

- fabrication,
- site moves,
- mating,
- sea transportation,
- installation,
- decommissioning.

Structural design covering marine operation, construction sequences shall be undertaken in accordance with NORSOK N-001.

Marine operations should be undertaken in accordance with the requirements stated in the VMO standard.

All marine operations shall, as far as practicable, be based upon well proven principles, techniques, systems and equipment and shall be undertaken by qualified, competent personnel possessing relevant experience.

Structural responses resulting from one temporary phase condition (e.g. a construction or transportation operation) that may affect design criteria in another phase shall be clearly documented and considered in all relevant design workings.

If it is intended to dry-dock the unit the bottom structure shall be suitably strengthened to withstand such actions.

Fabrication

Planning of construction sequences and of the methods of construction shall be performed. Actions occurring in fabrication phases shall be assessed and, when necessary the structure and the structural support arrangement shall be evaluated for structural adequacy.

Major lifting operations shall be evaluated to ensure that deformations are within acceptable levels and that relevant strength criteria are satisfied.

Mating

All relevant action effects incurred during mating operations shall be considered in the design process. Particular attention should be given to hydrostatic actions imposed during mating sequences.

Sea transportation

A detailed transportation assessment shall be undertaken which includes determination of the limiting environmental criteria, evaluation of intact and damage stability characteristics, motion response of the global system and the resulting, induced actions. The occurrence of slamming actions on the structure and the effects of fatigue during transport phases shall be evaluated when relevant.

Satisfactory compartmentation and stability during all floating operations shall be ensured.

All aspects of the transportation, including planning and procedures, preparations, sea-fastenings and marine operations should comply with the requirements of the warranty authority.

Installation

Installation procedures of foundations (e.g. drag embedded anchors, piles, suction anchor or dead weights etc.) shall consider relevant static and dynamic actions, including consideration of the maximum environmental conditions expected for the operations.

The actions induced by the marine spread involved in the operations and the forces exerted on the structures utilised in positioning the unit, such as fairleads and pad-eyes, shall be considered for local strength checks.

Decommissioning

Abandonment of the unit shall be planned for in the design stage. Decommissioning phases for column stabilised units are however not normally considered to provide design load conditions for the structure of a unit and may therefore normally be disregarded in the design phase.

M.2 Structural classification and material selection

M.2.1 Structural classification

Selection of steel quality, and requirements for inspection of welds, shall be based on a systematic classification of welded joints according to the structural significance and the complexity of the joints/connections as documented in Clause 5.

In addition to in-service operational phases, consideration shall be given to structural members and details utilised for temporary conditions, e.g. fabrication, lifting arrangements, towing arrangements, etc.

Basic considerations

Structural connections in column stabilised units designed in accordance with this annex should normally not fall within the categorisation criteria relevant for design class DC1 or DC2. In particular, relevant failure of a single weld, or element, should not lead to a situation where the accidental limit state condition is not satisfied. Structural connections will therefore be categorised within classification groups DC3, DC4 or DC5.

Consideration shall be given to address areas where through thickness tensile properties may be required.

Special consideration shall be given to ensure the appropriate inspection category for welds with high utilisation in fatigue if the coverage with standard local area allocation is insufficient.

Examples of typical design classes applicable to column stabilised units are stated below. These examples provide minimum requirements and are not intended to restrict the designer in applying more stringent requirements should such requirements be desirable.

Typical locations - Design class: DC3

Locations in way of connections at major structural interfaces (e.g. pontoon/pontoon, pontoon/column, column/deck, and, major brace connections) should generally be classified as DC3 due to the complexity of the connections and the general difficulty with respect to inspection and repair.

Local areas in way of highly stressed complex structural connections (e.g. in way of fairleads, riser supports, topside structures, or other similar locations with high complexity) should also be classified as DC3.

DC3 areas may be limited to local, highly stressed areas if the stress gradient at such connections is large.

Typical locations - Design class: DC4

Except as provided for in the description for DC3 and DC5 structural categorisation, connections appropriate to general stiffened plate fields, including connections of structural elements supporting the plate fields, e.g. girders and stiffeners, should normally be classified in Design Class DC4.

General brace structural connections, (e.g. butt welds in brace plate fields) should be designated DC4, as should connections of the main supporting structure for topside components and equipment.

Typical locations - Design class: DC5

All welds for non-main load bearing structural elements, including non-watertight bulkheads, mezzanine (tween) decks, and, deck superstructures, may be classified as DC5.

The same applies for welds for internal outfitting structures in general.

M.2.2 Material selection

Material specifications shall be established for all structural materials utilised in a column stabilised unit. Such materials shall be suitable for their intended purpose and have adequate properties in all relevant design conditions. Material selection shall be undertaken in accordance with the principles given in Clause 5.

When considering criteria appropriate to material grade selection, adequate consideration shall be given to all relevant phases in the life cycle of the unit. In this connection there may be conditions and criteria, other than those from the in-service, operational phase, that provide the design requirements in respect to the selection of material. Such criteria may, for example, be design temperature and/or stress levels during marine operations.

In the selection of material grades adequate consideration shall be given to the appropriateness of the design temperature including the definition of such. When considering design temperature related to material selection the applicability that standard NORSOK requirements to fabrication of steel structures (see NORSOK M-101) are based upon a minimum design temperature of -14 °C, shall be evaluated.

Selection of steel quality for structural components shall normally be based on the most stringent DC of the joints involving the component.

Consideration shall be given to the presence of tensile stress in the direction of the thickness of the plate when determining the appropriate steel quality.

M.2.3 Inspection categories

Welding, and the extent of non-destructive examination during fabrication, shall be in accordance with the requirements stipulated for the appropriate "inspection category" as defined in NORSOK M-101.

Determination of inspection category should be in accordance with the criteria given in Table 5.3 and Table 5.4.

Inspection categories determined in accordance with this NORSOK standard provide requirements to the minimum extent of required inspection. When considering the economic consequence that repair may entail, for example, in way of complex connections with limited or difficult access, it may be considered prudent engineering practice to require more demanding requirements to inspection than the required minimum.

When determining the extent of inspection, and the locations of required NDT, in addition to evaluating design parameters (for example fatigue utilisation) consideration should be given to relevant fabrication parameters including; positioning of block (section) connections, the possibility that manual welded connections may not achieve the quality that automatic weld locations achieve, etc.

The lower the extent of NDT selected, the more important is the representativeness of the NDT selected and performed. The designer should therefore exercise good engineering judgement in indicating mandatory locations for NDT, where variation of utility along welds is significant.

M.2.4 Guidance to minimum requirements

Figure M.2-1 to Figure M.2-3 illustrate minimum requirements to selection of DC, and inspection category for typical column stabilised unit, structural configurations.

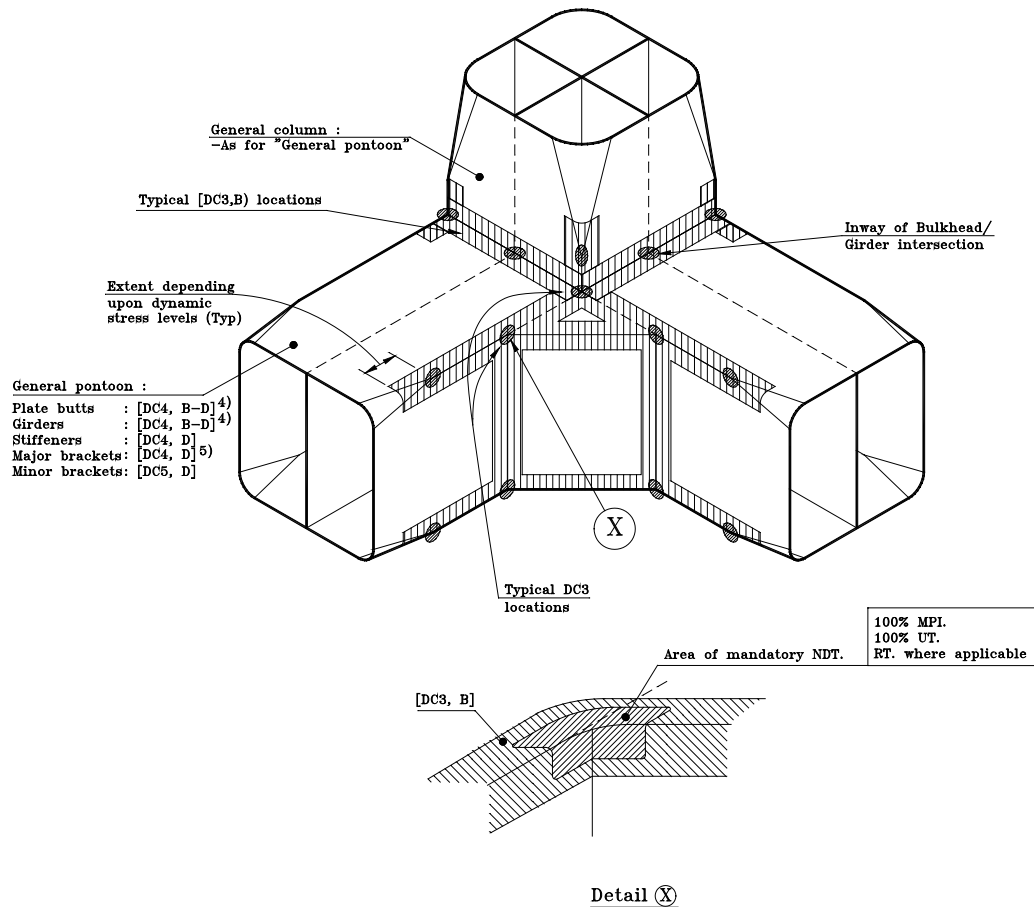


Figure M.2-1 Typical "ring" pontoon DCs and inspection categories

Key:

1. [DCn, ..] Design Class n, N
2. [....., N] Inspection Category, N

Notes:

1. In way of the pontoon/column connection (except in way of brackets), the pontoon deck plate fields will normally be the continuous material. These plate fields should normally be material with documented through-thickness properties (i.e. Steel Quality Level I material).
2. Shaded areas indicated in the figures are intended to be three-dimensional in extent. This implies that, in way of these locations, the shaded area logic is not only to apply to the outer surface of the connection but is also to extend into the structure.
3. "Major brackets" are considered, within the context of this figure, to be primary, load-bearing brackets supporting primary girders. "Minor brackets" are all other brackets.
4. The inspection categories for general pontoon, plate butt welds and girder welds to the pontoon shell are determined based upon, amongst others: accessibility, fatigue utilisation, and dominating stress direction (see NORSOK N-004, Table 5.3 and Table 5.4). E.g. Variations in dynamic stress levels across the section of the pontoon and also along the length of the pontoon may alter a general inspection category designation.
5. Major bracket toes will normally be designated as locations with a mandatory requirement to MPI.

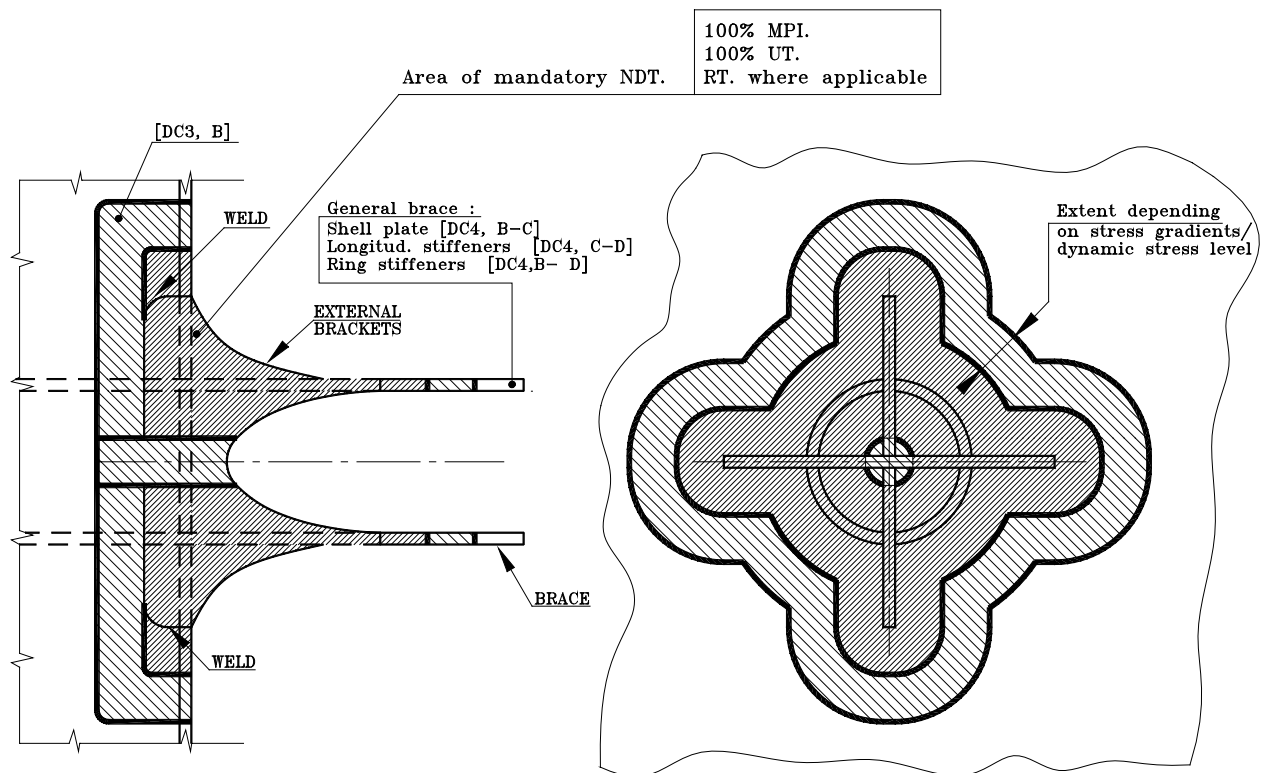


Figure M.2-2 Typical brace/column design classes and inspection categories

Key:

1. [DCn, ..] Design Class n, N
2. [....., N] Inspection Category, N

Notes:

1. In way of the column/brace connection the brace, and brace bracket plate fields, will normally be the continuous material. These plate fields should normally be material with documented through-thickness properties, i.e. Steel Quality Level I material.
2. Figure M.2-2 illustrates a relatively small brace connected to a column. For large brace connections the extent of the DC3 area may be limited somewhat to that illustrated.

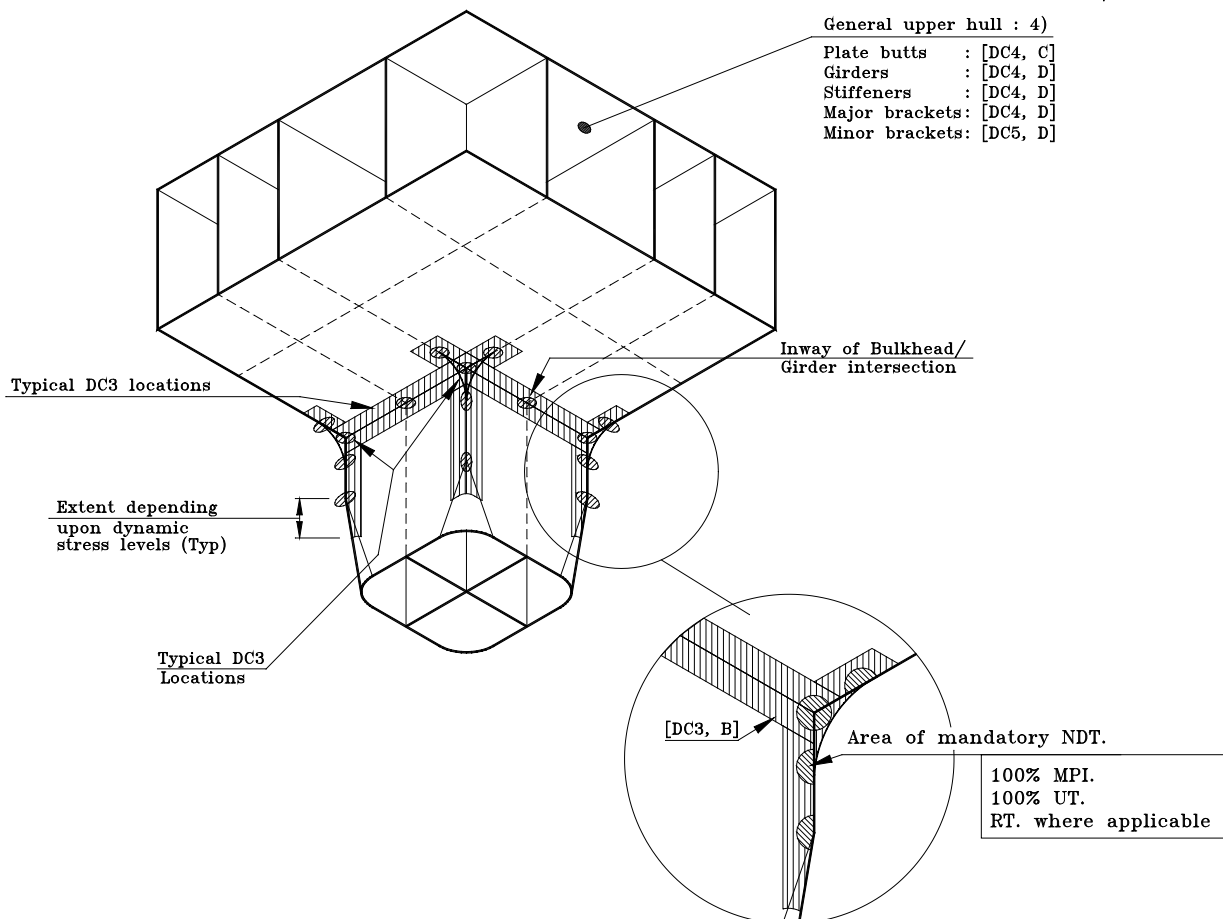


Figure M.2-3 Typical column/upper hull (deck box) design classes and inspection categories

Key:

1. [DCn, ..] Design Class n, N
2. [....., N] Inspection Category, N

Notes:

1. In way of the column/upper hull connection (except in way of brackets) the upper hull deck plate fields will normally be the continuous material. These plate fields should normally be material with documented through-thickness properties, i.e. Steel Quality Level I material.
2. Shaded areas indicated in the figures are intended to be three-dimensional in extent. This implies that, in way of these locations, the shaded area logic is not only to apply to the outer surface of the connection but is also to extend into the structure.
3. "Major brackets" are considered, within the context of this figure, to be primary, load-bearing brackets supporting primary girders. "Minor brackets" are all other brackets.
4. The inspection categories stated for general upper hull are determined based upon, the assumption that the fatigue utilisation may be categorised as being "low", i.e. that Table 5.3 is relevant. This assumption shall be verified.

M.3 Actions

M.3.1 General

Characteristic actions are to be used as reference actions. Design actions are, in general, defined in NORSOK N-003. Guidance concerning action categories relevant for column stabilised unit designs are given in the following.

M.3.2 Permanent actions

Permanent actions are actions that will not vary in magnitude, position, or direction during the period considered, and include

- "light mass" of the unit, including mass of permanently installed modules and equipment, such as accommodation, helideck, drilling and production equipment,
- hydrostatic pressures resulting from buoyancy,
- pretension in respect to mooring, drilling and production systems, e.g. mooring lines, risers etc..

M.3.3 Variable actions

Variable actions are actions that may vary in magnitude, position and direction during the period under consideration.

Except where analytical procedures or design specifications otherwise require, the value of the variable actions utilised in structural design should normally be taken as either the lower or upper design value, whichever gives the more unfavourable effect. Variable actions on deck areas for local, primary and global design are stated in NORSOK N-003, Table 5.1. The factor to be applied to "global design" actions (as stated in Table 5.1) is, however, not intended to limit the variable load carrying capacity of the unit in respect to non-relevant, variable load combinations. Global load cases should be established based upon "worst case", representative variable load combinations, where the limiting global design criteria is established by compliance with the requirements to intact and damage hydrostatic and hydrodynamic stability. See M.12 for a further discussion on this issue.

Global, design load conditions, and associated limiting criteria shall be documented in the stability manual for the unit /1/.

Variations in operational mass distributions (including variations in tank load conditions in pontoons) shall be adequately accounted for in the structural design.

Design criteria resulting from operational requirements should be fully considered. Examples of such operations may be

- drilling, production, workover, and combinations thereof,
- consumable re-supply procedures,
- maintenance procedures,
- possible mass re-distributions in extreme conditions.

Local, design deck actions shall be documented on a "load-plan". This plan shall clearly show the design uniform and concentrated actions for all deck areas, for each relevant mode of operation.

Dynamic actions resulting from flow through air pipes during filling operations shall be adequately considered in the design of tank structures.

Operational actions imposed by supply boats while approaching, mooring, or, lying alongside the unit should be considered in the design. If fendering is fitted, the combined fender/structural system is to be capable of absorbing the energy of such actions. The resulting response in the structural system is to satisfy the requirements of the considered limit state. A typical design impact action would, for example, be a SLS design consideration of a 5000 tonne displacement vessel with a contact speed of 0.5 m/s.

M.3.4 Deformation actions

Deformation actions are actions due to deformations applied to the structure.

Deformation actions resulting from construction procedures and sequences (e.g. as a result of mating operations, welding sequences, forced alignments of structure etc.) shall be adequately accounted for.

Other relevant deformation action effects may include those resulting from temperature gradients, as for example:

- when hot-oil is stored in a compartment adjacent to the sea;
- radiation from flaring operations.

M.3.5 Environmental actions

Environmental actions are actions caused by environmental phenomena.

Environmental conditions shall be described using relevant data, for the relevant period and areas in which the unit is to be fabricated, transported, installed and operated. All relevant environmental actions shall be considered.

Amongst those environmental actions to be considered, in the structural design of a column stabilised unit, are

- wave actions (including variable pressure, inertia, wave 'run-up', and slamming actions),
- wind actions,
- current actions,
- temperature actions,
- snow and ice actions.

Amongst those environmental action induced responses to be considered, in the structural design of a column stabilised unit, are

- dynamic stresses for all limit states (e.g. with respect to fatigue, see M.3.7),
- rigid body motion (e.g. in respect to air gap and maximum angles of inclination),
- sloshing,
- slamming induced vibrations,
- vortex induced vibrations (e.g. resulting from wind actions on structural elements in a flare tower),
- wear resulting from environmental actions at mooring and riser system interfaces with hull structures.

For column stabilised units with traditional, catenary mooring systems, earthquake actions may normally be ignored.

Further considerations with respect to environmental actions are given in NORSOK N-003.

M.3.6 Accidental actions

Accidental actions are actions related to abnormal operation or technical failure.

Risk analyses shall be undertaken to identify and assess accidental events that may occur and the consequences of such events, see Clause 11. Relevant, structural design events for the ALS condition are identified from the risk analyses. The following ALS events are amongst those required to be considered in respect to the structural design of a column stabilised unit:

- dropped objects (e.g. from crane handling);
- fire;
- explosion;
- collision;
- unintended flooding;
- abnormal wave events.

Further considerations in respect to accidental actions are given in Clause 11.

M.3.7 Repetitive actions

Repetitive actions, which may lead to possible significant fatigue damage, shall be evaluated. The following listed sources of fatigue actions shall, where relevant, be considered:

- waves (including those actions caused by slamming and variable (dynamic) pressures);
- wind (especially when vortex induced vibrations may occur);
- currents (especially when vortex induced vibrations may occur);
- mechanical vibration (e.g. caused by operation of machinery);
- mechanical loading/unloading (e.g. crane actions).

The effects of both local and global dynamic response shall be properly accounted for when determining response distributions related to fatigue actions.

M.3.8 Combination of actions

Load coefficients and load combinations for the design limit states are, in general, given in NORSOK N-001.

Structural strength shall be evaluated considering all relevant, realistic loading conditions and combinations. Scantlings shall be determined on the basis of criteria that combine, in a rational manner, the effects of relevant global and local responses for each individual structural element.

A sufficient number of load conditions shall be evaluated to ensure that the characteristic largest (or smallest) response, for the appropriate return period, has been established. For example, maximum global, characteristic responses for a column stabilised unit may occur in environmental conditions that are not associated with the characteristic, largest, wave height. In such cases, wave period and associated wave steepness parameters are more likely to be governing factors in the determination of maximum/minimum responses.

M.4 Ultimate limit states (ULS)

M.4.1 General

General considerations in respect to methods of analysis are given in NORSOK N-003, Clause 9.

Analytical models shall adequately describe the relevant properties of actions, stiffness, and displacement, and shall satisfactorily account for the local and system effects of time dependency, damping, and inertia.

It is normally not practical, in design analysis of column stabilised units, to include all relevant actions (both global and local) in a single model. Generally, a single model would not contain sufficient detail to establish local responses to the required accuracy, or to include consideration of all relevant actions and combinations of actions (see M.12 for a more detailed discussion of this item). It is often the case that it is more practical, and efficient, to analyse different action effects utilising a number of appropriate models and superimpose the responses from one model with the responses from another model in order to assess the total utilisation of the structure. The modelling guidance given in the following therefore considers a proposed division of models in accordance with an acceptable analytical procedure. The procedures described are not however intended to restrict a designer to a designated methodology when an alternative methodology provides for an acceptable degree of accuracy, and includes all relevant action effects. Further, the modelling procedures and guidance provided are intended for establishing responses to an acceptable level of accuracy for final design purposes. For preliminary design, simplified models are recommended to be utilised in order to more efficiently establish design responses and to achieve a simple overview of how the structure responds to the designing actions.

M.4.2 Global models

The intention of the global analysis model(s) should be to enable the assessment of responses resulting from global actions.

An example of a global analysis model is shown in Figure M.4-1.

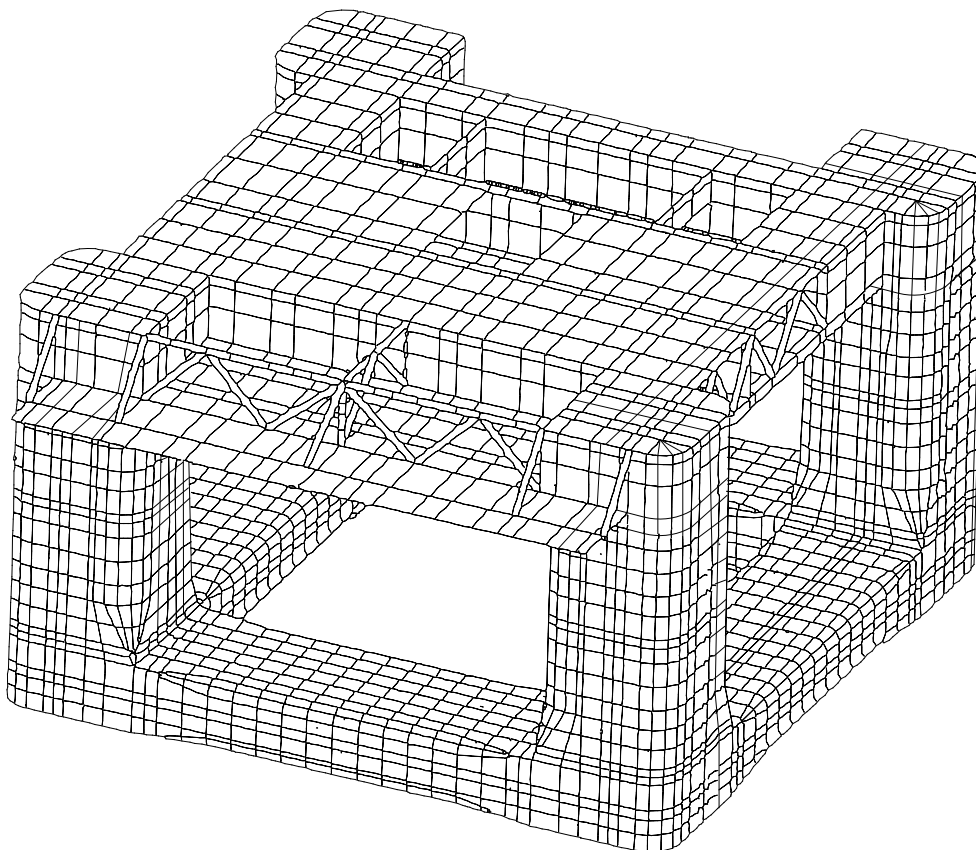


Figure M.4-1 Example of a global analysis model

M.4.2.1 Sea loading model

Global action effects

The environmental action model (e.g. simulating the appropriate wave conditions) shall be based upon site specific data in accordance with NORSOK N-003, Clause 6. The stochastic nature of wave actions shall be adequately accounted for.

For extreme response analysis, the short-crested nature of the sea may be accounted for according to the requirements given in NORSOK N-003, 6.2.2.3. Appropriate wave spectra criteria (as given in NORSOK N-003, 6.2.2.3) based upon site specific environmental data shall be considered when simulating sea actions.

A simple, Morrison element model may normally be utilised in order to ascertain global actions in preliminary design stages. It may be prudent to apply a contingency factor to the global actions resulting from such a model. A contingency factor of 1.2 to 1.3 would generally be considered as being appropriate.

For final design a diffraction model should be created in order to more accurately establish the design actions.

Global viscous damping effects may be included in the model either by the inclusion of Morrison elements, or by the inclusion of relevant terms in a global damping matrix. Viscous effects are non-linear and should therefore be carefully considered in frequency domain analyses when such effects may be considered as significantly affecting the response, e.g. in the evaluation of motion response (in connection with establishing air gap and acceleration parameters).

Where hydrostatic actions are modelled explicitly (as opposed to an implicit inclusion from a diffraction model) it shall be ensured that adequate account is given to the inclusion of end-pressure action effects, e.g. in respect to axial actions acting on pontoon structures.

Local viscous (drag) action effects

For relatively slender members, viscous (drag) local action effects should be accounted for. In such cases, the inclusion of local drag action effects may be undertaken in a number of ways, for example

- by use of hybrid (diffraction and Morrison) models of the structural elements,
- by only having a Morrison model of selected individual elements,
- by the inclusion of drag action effects by hand calculations.

M.4.2.2 Structural model

For large volume, thin-walled structures, three-dimensional finite element structural models created in shell (or membrane) finite elements, are normally required. For space frame structures consisting of slender members a three-dimensional, space frame representation of the structure may be considered adequate.

The stiffness of major structural connections (e.g. pontoon/column, column/deck) shall be modelled in sufficient detail to adequately represent the stiffness of the connection, such that the resulting responses are appropriate to the model being analysed.

It is preferable that actions resulting from the sea-loading model are mapped directly onto a structural model.

M.4.2.3 Mass modelling

A representative number of global design load conditions, simulating the static load distribution for each draught, should be evaluated in the global model. This may be achieved by the inclusion of a "mass model". The mass model may be an independent model or may be implicitly included in the structural (or sea action) model(s).

Usually, only a limited number of global load conditions are considered in the global analysis. The global model may not therefore adequately cover all "worst case" global load distributions for each individual structural element. Procedures shall be established to ensure that the most unfavourable load combinations have been accounted for in the design.

Pontoon tank loading conditions

In respect to global pontoon tank loading arrangements the maximum range of responses resulting from the most onerous, relevant, static load conditions shall be established. In order to assess the maximum range of stresses resulting from variations in pontoon tank loading conditions a simplified model of the structure may be created. This simplified model may typically be a space frame model of the unit.

Responses for relevant load combinations with full pontoon tank loading conditions (maximum "sagging" condition) and empty tank loading conditions (maximum "hogging" condition) shall be considered. For column stabilised units with non-box type, upper hull arrangements it may also be necessary to investigate conditions with non-symmetric, diagonally distributed tank loading conditions. Response resulting from

variations in pontoon tank load conditions may normally be considered as providing a significant contribution to the designing stress components in the areas of the upper and lower flanges of the pontoon deck and bottom plate fields. Such variations in stresses due to full/empty load combinations of pontoon tanks shall therefore be explicitly accounted for when considering logical combinations of global and local responses in the structural design.

M.4.2.4 Global analysis

Model testing shall be undertaken when non-linear effects cannot be adequately determined by direct calculations. In such cases, time domain analysis may also be considered as being necessary.

Where non-linear actions may be considered as being insignificant, or where such actions may be satisfactorily accounted for in a linear analysis, a frequency domain analysis can be undertaken. Transfer functions for structural response shall be established by analysis of an adequate number of wave directions, with an appropriate radial spacing. A sufficient number of periods shall be analysed to

- adequately cover the site specific wave conditions,
- satisfactorily describe transfer functions at, and around, the wave "cancellation" and "amplifying" periods,
- satisfactorily describe transfer functions at, and around, the heave resonance period of the unit.

Global, wave-frequency, structural responses shall be established by an appropriate methodology, for example:

- a regular wave analysis;
- a "design wave" analysis;
- a stochastic analysis.

These analytical methods are described in NORSOK N-003, 9.3.3.

Regular wave analysis:

A sufficient number of wave periods and wave directions shall be investigated to clearly demonstrate that maximum responses have been determined by the undertaken analysis. A wave directional radial spacing of maximum 15 degrees, should be utilised in a regular wave design analysis. However, it may be considered advisable to investigate with a spacing less than 15 degrees for structure with a substantial consequence of failure. Appropriate consideration shall be given to the modelling and inclusion of wave steepness characteristics.

Design wave analysis:

Design waves, utilised to simulate characteristic, global hydrodynamic responses shall be established based upon recognised analytical practices for column stabilised units. Appropriate consideration shall be given to the modelling and inclusion of wave steepness and spectral characteristics. A sufficient number of wave periods and wave directions shall be investigated to clearly demonstrate that maximum responses have been determined by the undertaken analysis. In design wave analysis a wave directional radial spacing of maximum 15 degrees or less, should normally be utilised.

The following characteristic responses will normally be governing for the global strength of a column stabilised unit:

- split ("squeeze and pry") force between pontoons;
- torsion ("pitch connecting") moment about a transverse horizontal axis;
- longitudinal shear force between the pontoons;
- combined longitudinal shear/split force between pontoons;
- longitudinal acceleration of deck mass;
- transverse acceleration of deck mass;
- vertical acceleration of deck mass.

Particular attention shall be given to the combined action effects of the characteristic responses of split and longitudinal shear force. Generally, it will be found that maximum longitudinal shear forces occur on a different wave heading than that heading providing maximum response when simultaneous split forces are taken into account.

Stochastic analysis:

Stochastic methods of analysis are, in principle recognised as providing the most accurate methods for simulating the irregular nature of wave actions. Long term, stress distributions shall be established based upon the relevant site specific, scatter diagram wave data.

In a stochastic design analysis, a wave directional radial spacing of 15 degrees or less, should be utilised.

Responses resulting from stochastic analytical procedures may however, not be particularly well suited to structural design as information concerning the simultaneity of internal force distribution is generally not available.

M.4.3 Local structural models

An adequate number of local structural models should be created in order to evaluate response of the structure to variations in local actions, e.g. in order to evaluate pressure acting across a structural section, lay-down loads acting on deck plate field, or, support point loads of heavy items of equipment.

The model(s) should be sufficiently detailed such that resulting responses are obtained to the required degree of accuracy. A number of local models may be required in order to fully evaluate local response at all relevant sections.

Example of local pressure acting on a pontoon section:

An example of considerations that should be evaluated in connection with the load case of local pressures acting on a pontoon section of a column stabilised unit are given below.

Typical parameters contributing to variations in internal and external local pressure distributions acting on the pontoon tanks of a cross-section of a column stabilised unit are illustrated in Figure M.4-2. For details concerning tank pressure actions reference should be made to NORSOK N-003, 5.4.

A local structural model should be created with the intention of simulating local structural response for the most unfavourable combination of relevant actions.

When evaluating local response from the local model the following considerations may apply:

- The intention of the local model is to simulate the local structural response for the most unfavourable combination of relevant actions. Relevant combinations of internal and external pressures for tanks should be considered (for both the intact and damage load conditions). Maximum (and minimum) design pressures (both internal and external) should be investigated.
- If cross-section arrangements change along the length of the structure, a number of local models may be required in order to fully evaluate local response at all relevant sections.
- When internal structural arrangements are such that the structure is divided across its section (e.g. there may be a watertight centreline bulkhead or access/pipe tunnel in the pontoon) relevant combinations of tank loadings across the section shall be considered.
- In most cases, for external plate fields, relevant combinations of external and internal pressure load cases will result in load conditions where the external and internal pressures are not considered to be acting simultaneously. For internal structures the simultaneous action of external and internal pressures load cases may provide for the governing design load case. For internal structures the internal pressure should normally not be considered to act simultaneously on both sides of the structure.
- Actions are usually applied in the analysis model at the girder level and not at the individual stiffener level in order to ensure that local stiffener bending is not included in the model response as the stiffener bending response is explicitly included in the buckling code check, see 6.5.
- For transversely stiffened structures (i.e. girders orientated in the transverse direction) the local responses extracted from the local model are normally responses in the structural transverse direction (σ_y in Figure M.4-3). For structural arrangements with continuous, longitudinal girder arrangements however a longitudinal response will also be of interest.
- For structural cross-sections without continuous longitudinal girder elements, two-dimensional structural models may be considered as being adequate.
- For space frame, beam models relevant consideration shall be given to shear lag effects.

Whilst this example considers the load case of local pressure acting on a pontoon section of a column stabilised unit, similar types of considerations are applicable in the evaluation of other local responses resulting from local actions on the column stabilised unit, e.g. lay-down actions.

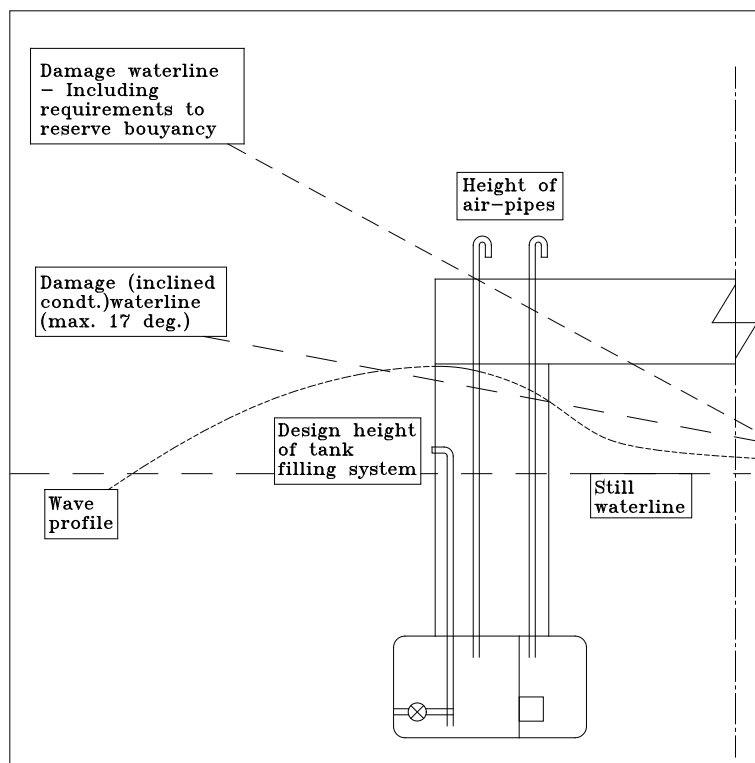


Figure M.4-2 Design action parameters acting as pressure on the pontoon tanks of a column stabilised unit

M.4.4 Superimposing responses

The simultaneity of the responses resulting from the analysis models described in M.4.2 and M.4.3 may normally be accounted for by linear superposition of the responses for logical load combinations.

Structural utilisation shall be evaluated in accordance with the requirements of Clause 6.

When evaluating responses by superimposing stresses resulting from a number of different models, consideration shall be given to the following:

- It should be ensured that responses from design actions are not included more than once. For example, when evaluating responses resulting from tank loading conditions (see M.4.2.3), it may be necessary to model a load condition, in the simplified model proposed in M.4.2.3, with a load case simulating that load case adopted in the global analysis model. In this way, stress variations resulting from relevant load conditions, when compared that load condition included in the global model may be established. Relevant peak responses may then be included in the capacity evaluation of the combined responses.
- Continuous, longitudinal structural elements (e.g. stiffened plate fields in the pontoon deck, bottom, sides, bulkheads, tunnels etc.) located outside areas of global stress concentrations, may be evaluated utilising linear superposition of the individual responses as illustrated in Figure M.4-3 for a pontoon section. The summation of such responses should then be evaluated against the relevant structural capacity criteria. In locations in way of column connections with pontoon and upper hull structures, global stress components become more dominant and should not be ignored.
- When transverse stress components are taken directly from the local structural model an evaluation, that it is relevant to ignore transverse responses from the global model, shall be carried out. This may normally be undertaken by considering the transverse stresses in the global model along the length of the structural element and ensuring that no additional transverse stress components have been set-up as a result of global stress concentrations, "skew" load conditions, or other interacting structural arrangements.
- Stiffener induced buckling failure normally tends to occur with lateral pressure on the stiffener side of the plate field. Plate induced buckling failure normally tends to occur with lateral pressure on the plate side.

Relevant combinations of buckling code checking should therefore include evaluation of the capacity with relevant lateral pressure applied independently to both sides of the plate field.

- In order to ensure that local bending stress components resulting from action acting directly on the stiffeners (σ_{bs} , σ_{bp} , -see Clause 6) are included in the buckling code check, the lateral pressure should be explicitly included in the capacity check. Unless, for the case in question, there is always a pressure acting over the stiffened plate field being evaluated, the capacity checking should include a buckling check with no lateral pressure in addition to the case with lateral pressure.

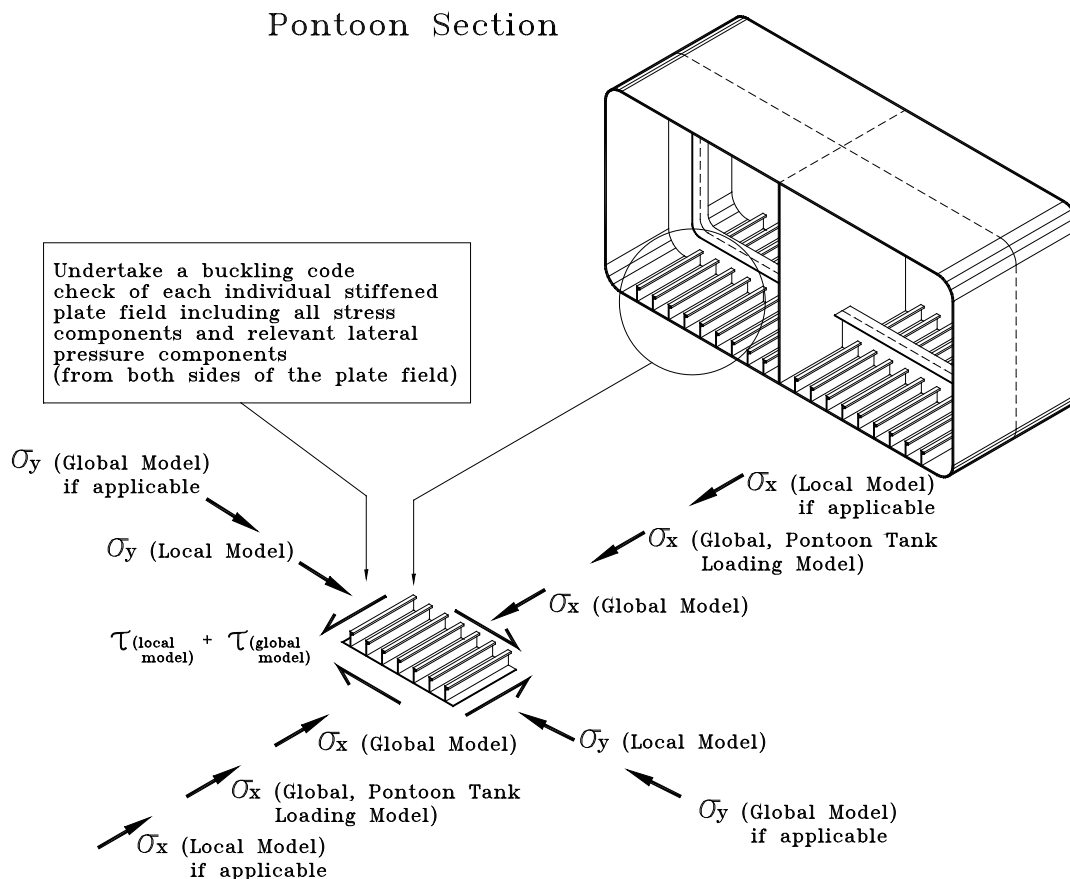


Figure M.4-3 Combination of stress components for buckling assessment of an individual stiffened plate field in a "typical" pontoon section.

M.5 Fatigue limit states (FLS)

M.5.1 General

General requirements and guidance concerning fatigue criteria are given in Clause 8, and DNV-RP-C203. Evaluation of the fatigue limit states shall include consideration of all significant actions contributing to fatigue damage both in non-operational and operational design conditions.

The minimum fatigue life of the unit (before the DFF is considered) should be based upon a period of time not being less than the planned life of the structure.

Assumptions related to the resistance parameters adopted in the fatigue design (e.g. with respect to corrosion protection) (see DNV-RP-C203) shall be consistent with the in-service structure.

Local effects, for example due to

- slamming,
- sloshing,
- vortex shedding,
- dynamic pressures,
- mooring and riser systems.

shall be included in the fatigue damage assessment, when relevant.

In the assessment of fatigue resistance, relevant consideration shall be given to the effects of stress raisers (concentrations) including those occurring as a result of

- fabrication tolerances (including due regard to tolerances in way of connections of large structural sections, e.g. as involved in mating sequences or section joints),
- cut-outs,
- details at connections of structural sections, e.g. cut-outs to facilitate construction welding.

M.5.2 Design fatigue factors (DFFs)

Criteria related to DFFs, are given in Clause 8.

Structural components and connections designed in accordance with this annex should normally satisfy the requirement to damage condition according to the ALS with failure in the actual joint defined as the damage. As such DFFs selected in accordance with Table 8-1 should normally fall within the classification: "Without substantial consequences".

When determining the appropriate DFF for a specific fatigue sensitive location, consideration shall be given to the following:

- Consideration that economic consequence of failure may indicate the use of larger design factors than those provided for as minimum factors.
- The categorisation: "Accessible/Above splash zone" is, intended to refer to fatigue sensitive locations where the possibility for close-up, detailed inspection in a dry and clean condition exists. If any of these requirements are not fulfilled, the relevant DFF should be considered as being that appropriate for "Accessible/Below splash zone", or, "No access or in the splash zone" as relevant to the location being considered.
- Evaluation of likely crack propagation paths (including direction and growth rate related to the inspection interval), may indicate the use of a different DFF than that which would be selected when the detail is considered in isolation, such that:
 - Where the likely crack propagation indicates that a fatigue failure, from a location satisfying the requirements for a "Non-substantial" consequence of failure, may result in a "Substantial" consequence of failure, such fatigue sensitive location is itself to be deemed to have a "Substantial" consequence of failure.
 - Where the likely crack propagation is from a location satisfying the requirement for a given "Access for inspection and repair" category to a structural element having another access categorisation, such location is itself to be deemed to have the same categorisation as the most demanding category when considering the most likely crack path. For example, a weld detail on the inside (dry space) of a submerged shell plate should be allocated the same DFF as that relevant for a similar weld located externally on the plate, see Figure M.5-1.

Implications of the above are that for internal structures, below or above splash zone, the DFF shall normally be 1 where access for inspection and repair is possible. Non-accessible areas, like welds covered with fire proofing and other areas inaccessible for inspection, shall be designed with DFF=3.

The hull shell below splash zone shall normally be assigned a DFF=2. This applies also to any attachment, internal or external, from where a fatigue crack can grow into the shell.

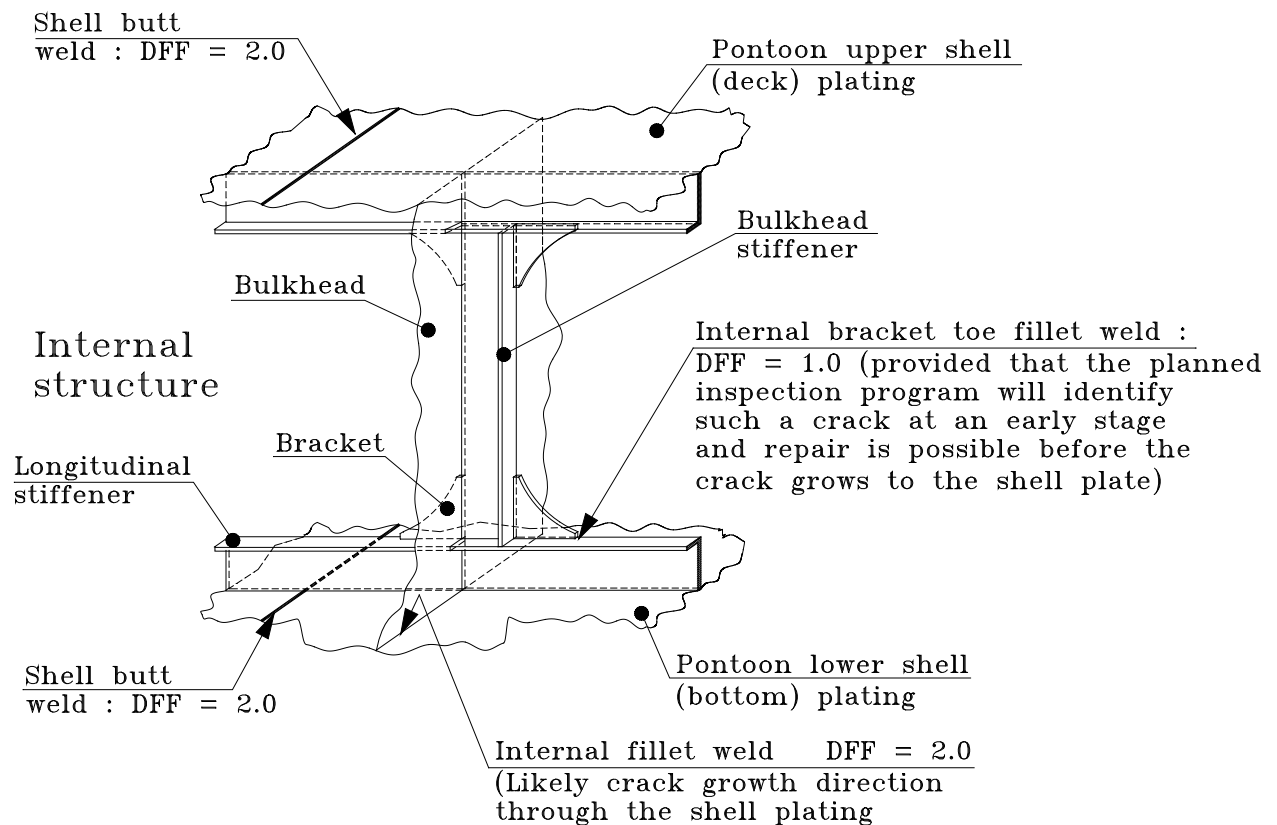


Figure M.5-1 Example illustrating considerations relevant for selection of DFFs in locations considered to have "non-substantial" consequence of failure

M.5.3 Splash zone

The definition of "splash zone" as given in NORSOK N-003, 6.6.3, relates to a highest and lowest tidal reference. For column stabilised units, for the evaluation of the fatigue limit state, reference to the tidal datum should be substituted by reference to the draught that is intended to be utilised when condition monitoring is to be undertaken. The requirement that the extent of the splash zone is to extend 5 m above and 4 m below this draught may then be applied. For application of requirements to corrosion addition, however, the normal operating draught should generally be considered as the reference datum.

If significant adjustment in draught of the unit is possible in order to provide for satisfactory accessibility in respect to inspection, maintenance and repair, account may be taken of this possibility in the determination of the DFFs. In such cases, however, a sufficient margin in respect to the minimum inspection draught should be considered when deciding upon the appropriate DFF in relation to the criteria for "Below splash zone" as opposed to "Above splash zone". Where draft adjustment possibilities exist, a reduced extent of splash zone may be applicable. See M.12 for further details.

The entire unit may be regarded as being above the splash zone if the unit is to be regularly dry-docked every 4 years to 5 years.

M.5.4 Fatigue analysis

M.5.4.1 General

The basis for determining the acceptability of fatigue resistance, with respect to wave actions, shall be appropriate stochastic fatigue analyses. The analyses shall be undertaken utilising relevant site specific environmental data and take appropriate consideration of both global and local (e.g. pressure fluctuation) dynamic responses. These responses do not necessarily have to be evaluated in the same model but the cumulative damage from all relevant effects should be considered when evaluating the total fatigue damage.

Simplified fatigue analyses may form the basis of a "screening" process to identify locations for which a detailed, stochastic fatigue analysis should be undertaken, e.g. at critical intersections. Such simplified fatigue analysis shall be calibrated, see M.5.4.2.

Local, detailed FE analysis of critical connections (e.g. pontoon/pontoon, pontoon/column, column/deck and brace connections) should be undertaken in order to identify local stress distributions, appropriate SCFs, and/or extrapolated stresses to be utilised in the fatigue evaluation, see DNV-RP-C203 for further details. Dynamic stress variations through the plate thickness shall be documented and considered in such evaluations.

Explicit account shall be taken of any local structural details that invalidate the general criteria utilised in the assessment of the fatigue strength. Such local details may, for example be access openings, cut-outs, penetrations etc. in structural elements.

Principal stresses (see DNV-RP-C203) should be utilised in the evaluation of fatigue responses.

M.5.4.2 Simplified fatigue analysis

Provided that the provisions stated in M.5.4.1, are satisfied, simplified fatigue analysis may be undertaken in order to establish the general acceptability of fatigue resistance. In all cases when a simplified fatigue analysis is utilised a control of the results of the simplified fatigue evaluation, compared to the stochastic results, shall be documented to ensure that the simplified analysis provides for a conservative assessment for all parts of the structure being considered.

Simplified fatigue analysis is particularly well suited for preliminary design evaluation studies. In such cases, a space frame model of the structure may be considered as being adequate. For final design however the basis model for the "screening" evaluation should be a model of similar (or better) detail to that described in M.4.2.2.

Simplified fatigue analyses should be undertaken utilising appropriate design parameters that provide for conservative results. Normally a two-parameter, Weibull distribution (see DNV-RP-C203) may be utilised to describe the long-term stress range distribution. In such cases the Weibull shape parameter ('h', see Equation (M.5.1)) should normally have a value between 1.0 and 1.1.

$$\Delta\sigma_{n_0} = \frac{(\ln n_0)^{\frac{1}{h}}}{(\text{DFF})^{\frac{1}{m}}} \left[\frac{\bar{a}}{n_0 \Gamma\left(1 + \frac{m}{h}\right)} \right]^{\frac{1}{m}} \quad (\text{M.5.1})$$

where

n_0	is the total number of stress variations during the lifetime of the structure
$\Delta\sigma_{n_0}$	is the extreme stress range that is exceeded once out of n_0 stress variations. The extreme stress amplitude $\Delta\sigma_{\text{ampl}_n}$ is thus given by $\left(\frac{\Delta\sigma_{n_0}}{2}\right)$.
h	is the shape parameter of the Weibull stress range distribution
$\bar{a}$	is the intercept of the design S-N curve with the log N axis (see, for example, DNV-RP-C203)
$\Gamma\left(1 + \frac{m}{h}\right)$	is the complete gamma function (see DNV-RP-C203)
m	is the inverse slope of the S-N curve (see DNV-RP-C203)
DFF	is the DFF

Generally, the simplified global fatigue analysis should consider the "F3", S-N class curve (see DNV-RP-C203), adjusted to include any thickness effect, as the minimum basis requirement. Areas not satisfying this

requirement are normally to be excluded from the simplified fatigue evaluation "screening" procedure. This implies that connections with a more demanding S-N fatigue class than F3, are not to be applied in the structure, e.g. if overlap connections are applied then the fatigue S-N class is to be suitably adjusted. The cumulative fatigue damage may then be obtained by considering the dynamic stress variation, $\Delta\sigma_{Actual(n_0)}$, which is exceeded once out of ' n_0 ' cycles [see Equation (M.5.2) with the allowable equivalent stress variation, $\Delta\sigma_{n_0}$, calculated from Equation (M.5.1). The fatigue life thus obtained is found from Equation (M.5.1)].

$$N_{Actual} = N_{Design} \left[\frac{\Delta\sigma_{n_0}}{\Delta\sigma_{Actual(n_0)}} \right]^m \quad (M.5.2)$$

where

- N_{Actual} is the actual (calculated) fatigue life
- N_{Design} is the target fatigue life
- $\Delta\sigma_{n_0}$ is described in Equation (M.5.1)
- $\Delta\sigma_{Actual(n_0)}$ is the actual design, dynamic stress variation which is exceeded once out of " n_0 " cycles.

When the simplified fatigue evaluation involves utilisation of the dynamic stress responses resulting from the global analysis (as described in M.4.2), the response should be suitably scaled to the return period of the basis, minimum fatigue life of the unit. In such cases, scaling may normally be undertaken utilising the appropriate factor found from Equation (M.5.3).

$$\Delta\sigma_{n_0} = \Delta\sigma_{n_{100}} \left[\frac{\log n_0}{\log n_{100}} \right]^{\frac{1}{h}} \quad (M.5.3)$$

where

- n_{100} is the number of stress variations (e.g. 100 years) appropriate to the global analysis
- $\Delta\sigma_{n_{100}}$ is the extreme stress range that is exceeded once out of n_{100} stress variations

[Other parameters are as for Equation (M.5.1) and Equation (M.5.2)].

M.5.4.3 Stochastic fatigue analysis

Stochastic fatigue analyses shall be based upon recognised procedures and principles utilising relevant site specific data.

Analyses shall include consideration of the directional probability of the environmental data. Providing that it can be satisfactorily documented, scatter diagram data may be considered as being directionally specific. Relevant wave spectra shall be utilised. Wave energy spreading may be taken into account if relevant. Structural response shall be determined based upon analyses of an adequate number of wave directions. Generally a maximum radial spacing of 15 degrees should be considered. Transfer functions should be established based upon consideration of a sufficient number of periods, such that the number, and values of the periods analysed

- adequately cover the site specific wave data,
- satisfactorily describe transfer functions at, and around, the wave "cancellation" and "amplifying" periods. Consideration should be given to take account that such "cancellation" and "amplifying" periods may be different for different elements within the structure),
- satisfactorily describe transfer functions at, and around, the relevant excitation periods of the structure.

Stochastic fatigue analyses utilising simplified structural model representations of the unit (e.g. a space frame model) may form the basis of a "screening" process to identify locations for which a, stochastic fatigue analysis, utilising a detailed model of the structure, should be undertaken (e.g. at critical intersections). When space frame, beam models are utilised in the assessment of the fatigue strength of large volume structures, (other than for preliminary design studies) the responses resulting from the beam models should be

calibrated against a more detailed model to ensure that global stress concentrations are included in the evaluation process.

M.6 Accidental limit states (ALS)

M.6.1 General

General guidance and requirements concerning accidental events are given in Clause 9, and Annex A.

Units shall be designed to be damage tolerant (i.e. credible accidental damages, or events, are not to cause loss of global structural integrity) see M.9.1. The capability of the structure to redistribute actions should be considered when designing the structure.

In the design phase, attention shall be given to layout and arrangements of facilities and equipment in order to minimise the adverse effects of accidental events.

Satisfactory protection against accidental damage may be obtained by a combination of the following principles:

- reduction of the probability of damage to an acceptable level;
- reduction of the consequences of damage to an acceptable level.

Structural design in respect to the ALS shall involve a two-stage procedure considering

- resistance of the structure to a relevant accidental event,
- capacity of the structure after an accidental event.

Global structural integrity shall be maintained both during and after an accidental event. Actions occurring at the time of a design accidental event and thereafter shall not cause complete structural collapse, or loss of hydrostatic (or hydrodynamic) stability, see M.8.

Requirements to compartmentation and stability in the damage condition are given in M.8.4. When the upper hull (deckbox) structure becomes buoyant in satisfying requirements to damage stability /1/, consideration shall be given to the structural response resulting from such actions.

M.6.2 Dropped objects

Critical areas for dropped objects shall be determined on the basis of the actual movement of potential dropped objects (e.g. crane actions) relative to the structure of the unit itself. Where a dropped object is a relevant accidental event, the impact energy shall be established and the structural consequences of the impact assessed.

The impact energy at sea level is normally not to be taken less than 5 MJ for cranes with maximum lifting capacity of more than 30 tonnes, however, reduced impact energy may be acceptable for smaller cranes and special purpose cranes.

The impact energy below sea level is assumed to be equal to the energy at sea level, unless otherwise documented.

Generally, dropped object assessment will involve the following considerations:

- assessment of the risk and consequences of dropped objects impacting topside, wellhead (e.g. on the seafloor), and safety systems and equipment. The assessment shall identify the necessity of any local structural reinforcement or protection to such arrangements;
- assessment of the risk and consequences of dropped objects impacting externally on the hull structure. However, the structural consequences are normally fully accounted for by the requirements for watertight compartmentation and damage stability (see M.8) and the requirement for structural redundancy of slender structural members, see M.9.1.

M.6.3 Fire

General guidance and requirements concerning accidental limit state events involving fire are given in Clause 9 and Annex A.

M.6.4 Explosion

In respect to design considering actions resulting from explosions one, or a combination of the following main design philosophies are relevant:

- ensure that the probability of explosion is reduced to a level where it is not required to be considered as a relevant design load case;
- ensure that hazardous locations are located in unconfined (open) locations and that sufficient shielding mechanisms (e.g. blast walls) are installed;

- locate hazardous areas in partially confined locations and design utilising the resulting, relatively small overpressures;
- locate hazardous areas in enclosed locations and install pressure relief mechanisms (e.g. blast panels) and design for the resulting overpressure.

As far as practicable, structural design accounting for large plate field rupture resulting from explosion actions should normally be avoided due to the uncertainties of the actions and the consequences of the rupture itself.

Structural support of blast walls, and the transmission of the blast action into main structural members shall be evaluated when relevant. Effectiveness of connections and the possible outcome from blast, such as flying debris, shall be considered.

M.6.5 Collision

Collision shall be considered as a relevant ALS load case for all structural elements of the unit that may be impacted in the event of a collision. The vertical zone of impact shall be based on the depth and draught of the colliding vessel and the relative motion of the vessel and the unit.

In the assessment of the collision condition, the following general considerations with respect to structural strength will normally apply:

- An evaluation shall be undertaken in order to assess the extent of structural damage occurring to the unit at the time of impact.
- The extent of the local damage resulting from the collision should be compared to that damage extent implicit in the NMD regulations covering damage stability of the unit /1/. Provided that the extent of local damage does not exceed that damage criteria stated in the NMD regulations, the unit shall satisfy the relevant damage stability requirements of the NMD, see also M.8.4. If the extent of the damage exceeds that damage criteria stated in the NMD regulations /1/, an equivalent level of safety to that implicit in the NMD stability regulations should be documented.
- NMD damaged condition requirements /1/, in respect to structural strength of watertight boundaries (including boundaries required for reserve buoyancy) shall be satisfied, see M.6.8.
- Global structural integrity of the unit after the collision shall be evaluated.
- Topsides structural arrangements should be designed for the damaged (inclined) condition.

Considerations concerning structural evaluation at, and after, the time of the collision are given below.

Damage occurring at the time of collision

A structural evaluation shall be performed in order to document the extent of the local damage occurring to the unit at the time of impact.

If a risk analysis shows that the greatest relevant accidental event with regard to collision is a drifting vessel at 2 m/s, with a displacement which does not exceed 5000 tonnes, the following kinetic energy occurring at the time of collision may be considered for the structural design:

- 14 MJ for sideways collision;
- 11 MJ for bow or stern collision.

Local damage assessment may be undertaken utilising sophisticated non-linear analytical tools, however, simplified analytical procedures will normally be considered as being sufficient to evaluate the extent of damage occurring under the action of the collision.

Simplified local damage assessment of the collision event normally involves the following considerations

- the typical geometry of the supply vessel, together with relevant force-deformation curves for side, bow and stern impact, documented in Annex A, may normally be utilised;
- in the local structural strength assessment the side, bow and stern profiles of the supply vessel are progressively "stepped" into the collision zone of the column stabilised unit, see example shown in Figure M.6-1;

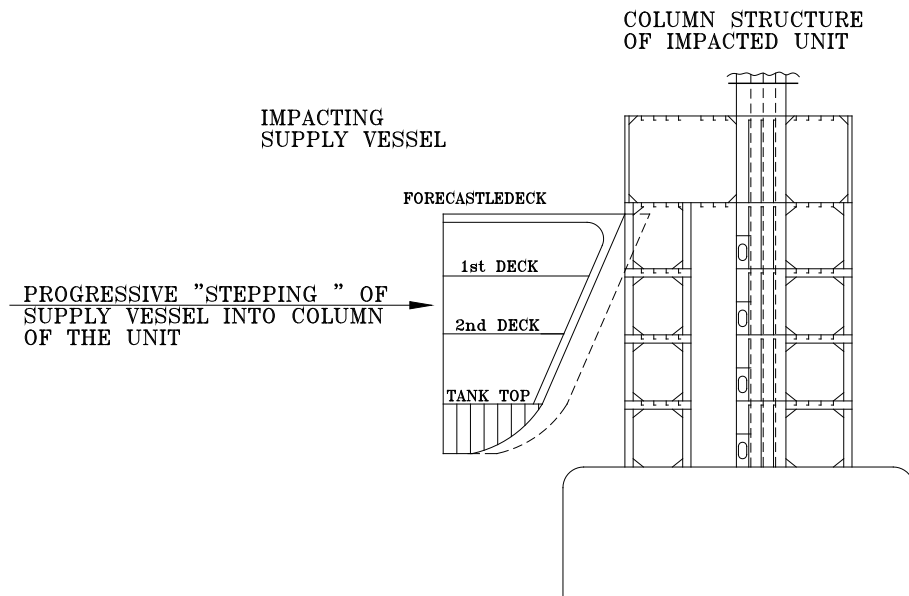


Figure M.6-1 Illustration of the bow profile of a supply vessel being "stepped" into the structure of a column stabilised unit

- by considering the local geometry of the supply vessel and the impacted structure, relevant force-deformation curves for the column stabilised unit may be produced;
- For a given action level the area under the force-deformation curves represents the absorption of energy. The distribution and extent of the damage results from the condition of equal collision force acting on the structures, and that the sum of the absorbed energies equals the portion of the impact energy dissipated as strain energy, i.e.

$$E_s + E_u = \int_0^{\Delta_s} P_s(\delta_s) d\delta_s + \int_0^{\Delta_u} P_u(\delta_u) d\delta_u \quad (\text{M.6.1})$$

where

P_s, P_u are the force in the impacting vessel and the impacted unit respectively,

δ_s, δ_u are the deformations in the impacting vessel and the impacted unit respectively.

This procedure is illustrated in Figure M.6-2.

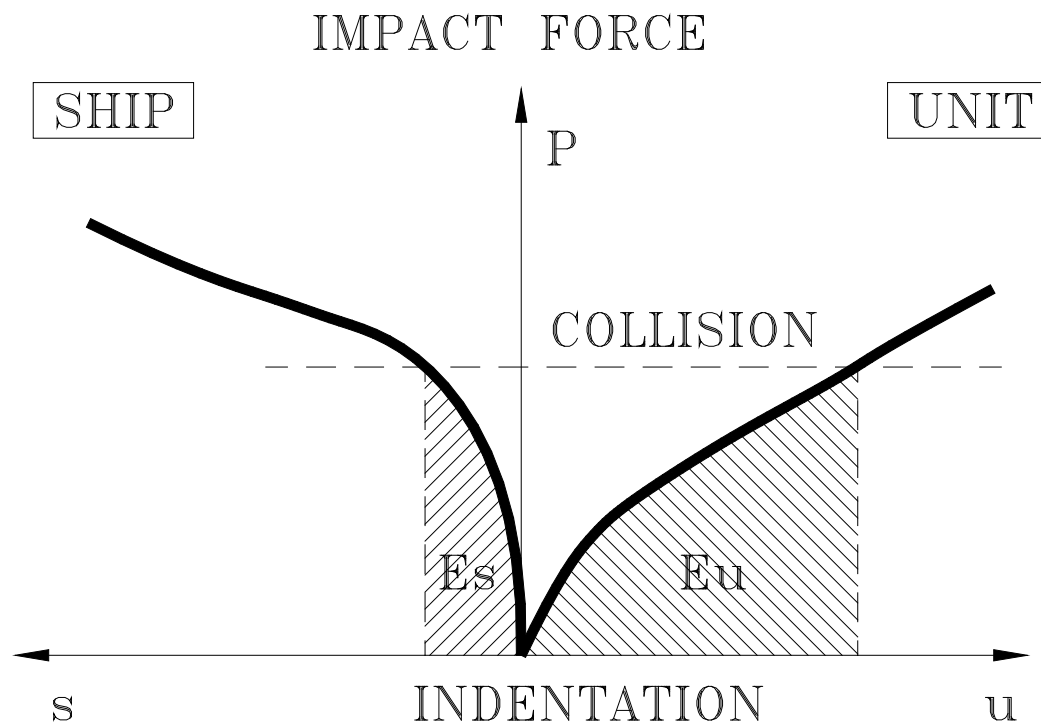


Figure M.6-2 Dissipation of strain energy global structural integrity after collision

Having evaluated the extent of local damage incurred by the relevant collision event (as described above) an assessment of the resulting, global structural capacity (considering environmental actions) shall be undertaken. In such an evaluation the following listed items are relevant:

- In cases where the impact damage is limited to local damage to the column particular consideration should be given to column/deckbox interfaces, and the damaged (impacted) section. For typical column structures a simplified approach to assess the reduced capacity of the structure in way of the damaged location would be to assume that all the impacted (deformed) area is ineffective.
- When counter-flooding is utilised as a means of righting the unit in an accidental event the actions resulting from such counter flooding shall be evaluated.
- In cases where the impact damage is limited to local damage to a single brace, redundancy requirements given in M.9.1 should adequately cover the required structural evaluation.
- NMD requirements to watertight boundary, structural strength in the damaged condition (see M.6.8) (including inclined angles resulting from requirements to reserve buoyancy) should be satisfied.
- In order to avoid risk assessment considerations in respect to implications of structural failure of topside structures in the inclined condition (e.g. in respect to the possibility of progressive collapse in the event of structural failure), it is normal practice to consider the structural capacity of topside structural arrangements in the damaged condition.

Capacity exceedances may be accepted for local areas provided that adequate account is given to the redistribution of forces.

Due to the large heel angles in the damaged condition, the in-plane force component of the deck box mass may be considerable. Normally part of the deck will be submerged and counteract this force. The most critical condition is therefore generally the heel angle corresponding to a water plane just below the deck box corner.

M.6.6 Unintended flooding

Considerations in respect to unintended flooding are generally system and stability design considerations rather than structural design. Requirements with respect to intact and reserve buoyancy conditions are normally considered to adequately cover any structural strength requirements (see M.6.8) in respect to unintended flooding.

M.6.7 Abnormal wave events

Air gap considerations with respect to evaluation of the abnormal wave events are given in M.7. Should any part of the structure receive wave impact actions in the abnormal wave condition the consequences of such wave impacts shall be evaluated.

M.6.8 Reserve buoyancy

Structural strength of watertight boundaries (both internal and external watertight boundaries, including all stiffeners and girders supporting the plate fields) shall comply with the requirements stated in NMD regulations /1/.

M.7 Air gap

M.7.1 General

Requirements and guidance to air gap analyses for a column stabilised unit are given in NORSOK N-003, 9.2.5.4. Both ULS (10^{-2}) and ALS (10^{-4}) conditions shall be evaluated.

In the ULS condition, positive air gap should be ensured. However, wave impact may be permitted to occur provided that it can be demonstrated that such actions are adequately accounted for in the design and that safety to personnel is not significantly impaired. See M.12 for more details.

Analysis undertaken to document air gap should be calibrated against relevant model test results. Such analysis shall include relevant account of

- wave/structure interaction effects,
- wave asymmetry effects,
- global rigid body motions (including dynamic effects),
- effects of interacting systems (e.g. mooring and riser systems),
- maximum/minimum draughts.

Column "run-up" action effects shall be accounted for in the design of the structural arrangement in way of the column/deck box connection. These "run-up" actions are to be treated as an environmental load component. However, they need not normally be considered as occurring simultaneously with other environmental responses. For further considerations in respect to run-up effects, reference should be made to NORSOK N-003.

Evaluation of air gap adequacy shall include consideration of all affected structural items including lifeboat platforms, riser balconies, overhanging deck modulus etc..

M.8 Compartmentation and stability

M.8.1 General

In the assessment of compartmentation and stability of a column stabilised unit, consideration shall be given to all relevant detrimental effects, including those resulting from

- environmental actions,
- relevant damage scenarios,
- rigid body motions,
- the effects of free-surface,
- boundary interactions (e.g. mooring and riser systems).

The mass distribution of a column stabilised unit, including all associated permanent and variable actions, shall be determined to an appropriate accuracy by a relevant procedure. An inclining test should be conducted when construction is as near to completion as practical in order to accurately determine the unit's mass and position of the centre of gravity. Changes in load conditions after the inclining test should be carefully accounted for.

Monitoring of the mass and centre of gravity shall be performed during the entire life cycle of the unit. A procedure for control of the mass and position of the centre of gravity of the unit shall be incorporated into the design.

Note: Special provisions relating to the Norwegian activities

1. Special provisions relating to subdivision, stability and freeboard for units engaged in Norwegian petroleum activities are given in NORSOK N-001, 7.10.

M.8.2 Compartmentation and watertight integrity

Compartmentation, and watertight and weather tight integrity of a column stabilised unit shall satisfy the general requirements as stated in ISO 19900. Detailed provisions stated in IMO MODU Code /2/, should also be satisfied.

The number of openings in watertight structural boundaries shall be kept to a minimum. Where penetrations are necessary for access, piping, venting, cables etc., arrangements shall be made to ensure that the watertight integrity of the structure is maintained.

Where individual lines, ducts or piping systems serve more than one watertight compartment, or are within the extent of damage resulting from a relevant accidental event, arrangements shall be provided to ensure that progressive flooding will not occur.

M.8.3 Stability

Stability of a column stabilised unit in the operational phase, shall satisfy the requirements of the relevant provisions stated in the IMO MODU Code /2/.

Adequacy of stability shall be established for all relevant operational and temporary phase conditions. The assessment of stability shall include consideration of both the intact and damaged conditions. It shall be ensured that the assumed basis for the damage stability design criteria is compatible with accidental events identified as being relevant for the structure, see M.6 and M.8.4.

M.8.4 Damaged condition

The dimensioning extent of damage and loss of buoyancy shall be based on a risk analysis, see M.6.1. If such risk analysis shows that the greatest relevant accidental event with regard to collision is a drifting vessel with a displacement that does not exceed 5000 tonnes, the extent of damage indicated in the Norwegian Maritime Directorates, Regs. for Mobile Offshore Units, 1991, /1/, may be utilised.

In order to ensure adequacy of ballast systems, both with respect to unintentional flooding, (see M.6.6) and capacity in the damage condition (see M.6.5), the requirements to ballast systems, stated in the Norwegian Maritime Directorate, Regs. for Mobile Offshore Units, 1991, /1/, concerning ballast systems on mobile offshore units, /3/, Section 11 and Section 12 should be satisfied.

M.9 Special considerations

M.9.1 Structural redundancy

Structural robustness shall, when considered necessary, be demonstrated by appropriate analysis. Slender, main load bearing, structural elements shall normally be demonstrated to be redundant in the accidental limit state condition.

Considerations with respect to redundancy of bracing systems are given in M.9.2.

M.9.2 Brace arrangements

For bracing systems (see M.12, Comm. M.9.1) the following listed considerations shall apply:

- brace structural arrangements shall be investigated for relevant combinations of global and local actions;
- structural redundancy of slender bracing systems (see M.9.1) shall normally include brace node redundancy (i.e. all bracings entering the node), in addition to individual brace element redundancy;
- brace end connections (e.g. brace/column connections) shall normally be designed such that the brace element itself will fail before the end connection;
- underwater braces should be watertight and have a leakage detection system to make it possible to detect cracking at an early stage;
- when relevant (e.g. in the self-floating, transit condition) the effect of slamming on braces shall be considered.

M.9.3 Structure in way of a fixed mooring system

Local structure in way of fairleads, winches, etc. forming part of the position mooring system is, as a minimum, to be capable of withstanding forces equivalent to 1.25 times the breaking strength of the individual mooring line. The strength evaluation should be undertaken utilising the most unfavourable operational direction of the anchor line. In the evaluation of the most unfavourable direction, account shall be taken of relative angular motion of the unit in addition to possible line lead directions.

M.9.4 Structural detailing

In the design phase particular attention should be given to structural detailing, and requirements to reinforcement in areas that may be subjected to high local stresses, for example:

- design class 3 (DC3) connections (see M.2.1);
- locations that may be subjected to wave impact (including column run-up actions);
- locations in way of mooring arrangements;
- locations that may be subjected to accidental, or operational, damage.

In way of DC3 connections, continuity of strength is normally to be maintained through joints with the axial stiffening members and shear web plates being made continuous. Particular attention should be given to weld detailing and geometric form at the point of the intersections of the continuous plate fields with the intersecting structure.

M.10 Documentation requirements for the "design basis" and "design brief"

M.10.1 General

Adequate planning shall be undertaken in the initial stages of the design process in order to obtain a workable and economic structural solution to perform the desired function. As an integral part of this planning, documentation shall be produced identifying design criteria and describing procedures to be adopted in the structural design of the unit.

Applicable codes, standards and regulations should be identified at the commencement of the design.

A summary document containing all relevant data from the design and fabrication phase shall be produced.

Design documentation (see below) shall, as far as practicable, be concise, non-voluminous, and, should include all relevant information for all relevant phases of the lifetime of the unit.

General requirements to documentation relevant for structural design are given in NORSOK N-001, Clause 5.

M.10.2 Design basis

A design basis document shall be created in the initial stages of the design process to document the basis criteria to be applied in the structural design of the unit.

A summary of those items normally to be included in the design basis document is included below.

Unit description and main dimensions

A summary description of the unit, including

- general description of the unit (including main dimensions and draughts),
- main drawings including:
 - main structure drawings (this information may not be available in the initial stage of the design),
 - general arrangement drawings,
 - plan of structural categorisation,
 - load plan(s), showing description of deck uniform (laydown) actions,
 - capacity (tank) plan.
- service life of unit,
- position keeping system description.

Rules, regulations and codes

A list of all relevant, and applicable, rules, regulations and codes (including revisions and dates), to be utilised in the design process.

Environmental design criteria

Environmental design criteria (including all relevant parameters) for all relevant conditions, including

- wind, wave, current, snow and ice description for 10^{-1} , 10^{-2} and 10^{-4} annual probability events,
- design temperatures.

Stability and compartmentation

Stability and compartmentation design criteria for all relevant conditions including

- external and internal watertight integrity plan,
- lightweight breakdown report,
- design loadcase(s) including global mass distribution,
- damage condition waterlines (this information may not be available in the initial stage of the design).

Temporary phases

Design criteria for all relevant temporary phase conditions including, as relevant:

- limiting permanent, variable, environmental and deformation action criteria;
- procedures associated with construction (including major lifting operations);
- essential design parameters associated with temporary phases (e.g. for mating load cases, mating weld-up sequences, crushing tube stiffness, for transit phases, transit speed etc.),
- relevant ALS criteria.

Operational design criteria

Design criteria for all relevant operational phase conditions including

- limiting permanent, variable, environmental and deformation action criteria,
- designing accidental event criteria (e.g. collision criteria),
- tank loading criteria (all tanks) including a description of system, with
 - loading arrangements,
 - height of air pipes,
 - loading dynamics,
 - densities.
- mooring actions.

Air gap

Relevant basis information necessary for the assessment of air gap sufficiency, including

- a description of the requirements to be applied in the ULS and ALS conditions,
- basis model test report (this information may not be available in the initial stage of the design).

In-service inspection criteria

A description of the in-service inspection criteria and general philosophy (as relevant for evaluating fatigue allowable cumulative damage ratios).

Miscellaneous

A general description of other essential design information, including

- description of corrosion allowances, where applicable.

M.10.3 Design brief

A design brief document shall be created in the initial stages of the design process. The purpose of the design brief shall be to document the intended procedures to be adopted in the structural design of the unit. All applicable limit states for all relevant temporary and operational design conditions shall be considered in the design brief.

A summary of those items normally to be included in the design brief document is included below.

Analysis models

A general description of models to be utilised, including description of

- global analysis model(s),
- local analysis model(s),
- load cases to be analysed.

Analysis procedures

A general description of analytical procedures to be utilised including description of procedures to be adopted in respect to

- the evaluation of temporary conditions,
- the consideration of accidental events,

- the evaluation of fatigue actions,
- air gap evaluation (including locations to be considered, damping, inclusion of asymmetry factors, disturbed (radiated) wave considerations, combined motion response),
- the establishment of dynamic responses (including methodology, factors, and relevant parameters),
- the inclusion of "built-in" stresses,
- the consideration of local responses (e.g. those resulting from mooring and riser actions, ballast distribution in pontoon tanks etc.),
- the consideration of structural redundancy.

Structural evaluation

A general description of the evaluation process, including

- description of procedures to be utilised for considering global and local responses,
- description of fatigue evaluation procedures (including use of DFFs, SN-curves, basis for stress concentration factors (SCFs), etc.),
- description of procedures to be utilised for code checking.

Air gap evaluation

A general description of the air gap evaluation procedure, including

- description of procedures to be utilised for considering air gap sufficiency.

M.11 References

- /1/ Regulations of 20 Dec. 1991, No.878, concerning stability, watertight subdivision and watertight/weathertight closing means on mobile offshore units, Norwegian Maritime Directorate.
- /2/ Code for the Construction and Equipment of Mobile Offshore Drilling Units, 1989, (1989 MODU CODE), International Maritime Organisation, London 1990.
- /3/ Regulations of 20 Dec. 1991, No.879, concerning ballast systems on mobile offshore units, Norwegian Maritime Directorate.
- /4/ 'Offshore Installations: Guidance on design, construction and certification', U.K., Health & Safety Executive, Fourth Edition, HMSO.

M.12 Commentary

Comm. M.1.1 General

The content of this annex is applicable to column stabilised units located at a single site over a prolonged period of time. Normally such units will be engaged in production activities. There may therefore be certain requirements and guidance given within this annex that are not considered as being fully appropriate for "mobile", column stabilised units, e.g. engaged in exploration drilling activities at a location for only a limited period of time.

Comm. M.3.3 Variable actions

Applicable variable actions acting on deck areas are stated in NORSOK N-003, Table 5.1. This table provides appropriate action factors for local, primary and global design. For floating units, full application of the global design action factors will lead to a load condition that, in practice, would not be possible to achieve as the total static mass is always required to be in balance with the buoyancy actions. Additionally requirements to stability and compartmentation will further restrict the practical distribution of variable actions.

In the assessment of global structural response the variable action design factors stated in Table 5.1 are not intended to limit the variable load carrying capacity of the unit in respect to non-relevant, variable load combinations. Full application of the global design factors, stated in Table 5.1, to all variable actions would normally imply such a restriction. Global design load conditions should therefore be established based upon "worst case", representative variable load combinations, where the limiting global mass distribution criteria is established taking into account compliance with the requirements to intact and damage hydrostatic and hydrodynamic stability.

These limit global load conditions shall be fully documented in the design basis, see M.10.2.

Comm. M.4.2 Global models

It is normally not practical to consider all relevant actions (both global and local) in a single model, due to, for example the following listed reasons:

- Single model solutions do not normally contain sufficient structural detailing (for ULS structural assessment, response down to the level of the stress in plate fields between stiffeners is normally required). Examples of insufficient structural detailing may be:
 - internal structure is not modelled in sufficient detail to establish internal structural response to the degree of accuracy required,
 - element type, shape or fineness (e.g. mesh size) is insufficient.
- Single model solutions do not normally account for the full range of internal and external pressure combinations. Examples of effects that may typically not be fully accounted for include:
 - internal tank pressure up to the maximum design pressure,
 - maximum external pressures (e.g. if a "design wave" analytical approach has been adopted, the maximum external pressure height is that height resulting from the design wave, which is not normally the maximum external pressure that the section may be subjected to),
 - the full extent of internal and external pressure combinations,
 - variations in tank loadings across the section of the pontoon (e.g. if the pontoon is sub-divided into a number of watertight compartments across its section),
 - load conditions that may not be covered by the global analysis (e.g. damage, inclined conditions).
- Single model solutions do not normally account for the full range of "global" tank loading conditions. Examples of global tank loading conditions that may typically not be fully accounted for include
 - tank loading distributions along the length of the pontoon,
 - asymmetric tank loadings from one pontoon as compared to another.
- Single model solutions may not fully account for all action effects. Examples of load effects that may typically not be fully accounted for include
 - viscous effects (drag loading) on slender members,
 - riser interface actions,
 - thruster actions.

Generally, single model solutions that do contain sufficient detail to include consideration of all relevant actions and load combinations are normally extremely large models, with a very large number of load cases. It is therefore often the case that it is more practical, and efficient, to analyse different action effects utilising a number of appropriate models and superimposing the responses from one model with the responses from another model in order to assess the total utilisation of the structure.

Comm. M.5.2 Design fatigue factors (DFFs)

If significant adjustment in draught is possible in order to provide for satisfactory accessibility in respect to inspection, maintenance and repair, a sufficient margin in respect to the minimum inspection draught should be considered when deciding upon the appropriate DFFs. As a minimum this margin is to be at least 1 m. However, it is recommended that a larger value be considered especially in the early design stages where sufficient reserve should be allowed for to account for design changes in the mass and the centre of mass of the unit. Consideration should further be given to operational requirements that may limit the possibility for ballasting/deballasting operations.

When considering utilisation of ROV inspection consideration should be given to the limitations imposed on such inspection by the action of water particle motion, e.g. waves. The practicality of such a consideration may be that effective underwater inspection by ROV, in normal sea conditions, may not be achievable unless the inspection depth is at least 10 m below the sea surface.

Comm. M.6.5 Collision

In the damaged, inclined condition (i.e. after the collision event) the structure shall continue to resist the defined environmental conditions, i.e. 10^{-2} annual probability of exceedance criteria. In practice, it is not normally considered practicable to analyse this damaged, inclined condition as the deck box structure becomes buoyant (due both to the static angle of inclination in the damaged condition and also due to rigid

body motion of the unit itself). The global system of loading and response becomes extremely non-linear. Even with exhaustive model testing it would be difficult to ensure that all relevant responses are measured, for all designing sea-state conditions, and directions, for all relevant damage waterlines.

Further, it is considered to be extremely unlikely that a 10^{-2} environmental load event would occur before corrective action had taken place, e.g. righting of the unit to an even keel. NMD requirements to ballast systems for column stabilised units, /3/, (and required in M.8.4) include requirements that the ballast system shall be capable of restoring the unit to an upright position, and an acceptable draught as regards strength, within 3 h after the collision damage event. On the assumption that the 5000 tonne, 2 m/s, collision criteria is based upon a free drifting supply vessel with a drift velocity equal to $H_s/2$ (where H_s = significant wave height), within a 3 h period, a 4 m significant wave height may be expected to increase to max. 6.5 m significant wave height. Hence, disregarding the damaged structural element itself, the ULS load case b. may generally be considered as providing more demanding design criteria than the ALS condition with 100-year load event. Additionally, in respect to global response, as soon as the deck box starts to become buoyant the global actions resulting from the inclined deck box mass rapidly become reduced.

With background in the above logic simplified "engineering" solutions to structural design for the collision event, that do not explicitly require analyses involving 100-year environmental actions in the inclined (heeled) condition, are normally considered as being acceptable.

Comm. M.7.1 General

In the ULS condition positive clearance between the upper hull (deck box) structure and the wave crest, including relative motion and interaction effects, should normally to be ensured. Localised, negative air gap, may however be considered as being acceptable for overhanging structures and appendages to the upper hull. In such cases full account of the wave impact forces is to be taken into account in the design. The consequence of wave impact shall not result in failure of a safety related system, e.g. lifeboat arrangements. It is recommended in the design phase to consider operational aspects, including requirements to inspection and maintenance, which may impose criteria to air gap that exceed minimum requirements.

In the context of M.7, column run-up actions are not considered as resulting in negative air-gap responses.

Comm. M.9.1 Structural redundancy

The requirement concerning structural redundancy of a slender structural element is included within this annex for the following listed reasons:

- in order to maintain compatibility with the requirements to column stabilised stated by the International Association of Classification Societies;
- in order to ensure that the structure exhibits ductility after first failure;
- in order to ensure that a single failure does not lead to a critical condition. A critical condition may for example be a fatigue failure that has not been accounted for in design calculations due to the fact that, in the process of fabrication, details built into the structure do not exactly reflect those details that considered in the assessment of the design.

Typically a member is to be considered as being slender if the reduced (relative) slenderness, $\bar{\lambda}$, is greater than 0.2, see 5.4.

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DESIGN OF STEEL STRUCTURES
ANNEX N
SPECIAL DESIGN PROVISIONS FOR TENSION LEG PLATFORMS

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N.1 INTRODUCTION

N.1.1 General

This Annex provides requirements and guidance to the structural design of tension leg platforms, constructed in steel, in accordance with the provisions of this NORSOK standard.

The requirements and guidance documented in this Annex are generally applicable to all configurations of tension leg platforms.

For novel designs, or unproved applications of designs where limited or no direct experience exists, relevant analyses and model testing, shall be performed which clearly demonstrate that an acceptable level of safety is obtained which is not inferior to the safety level set forth by this Annex when applied to traditional designs.

N.1.2 Definitions

Terms

A Tension Leg Platform (TLP) is defined as a buoyant installation connected to a fixed foundation by pretensioned tendons. The tendons are normally parallel, near vertical elements, acting in tension, which restrain the motions of the platform in heavy, pitch and roll. The platform is compliant in surge, sway and yaw.

The TLP tendon system comprises all components associated with the mooring system between, and including the top connection(s) to the hull and the bottom connection(s) to the foundation(s). Guidelines, control lines, umbilicals etc. for tendon service and/or installation are considered to be included as part of the tendon system.

The TLP foundation is defined as those installations at, or in, the seafloor which serve as anchoring of the tendons and provides transfer of tendon actions to the foundation soil.

The TLP hull consists of buoyant columns, pontoons and intermediate structural bracings, as applicable.

The TLP deck structure is the structural arrangement provided for supporting the topside equipment. Normally, the deck serves the purpose of being the major structural components to ensure pontoons, columns and deck acting as one structural unit to resist environmental and gravity actions.

Ringing is defined as the high frequency resonant response induced by transient loads from high, steep waves.

Springing is defined as the high frequency resonant response induced by cyclic (steady state) in low to moderate seastates.

High Frequency (HF) responses are defined as TLP rigid body motions at, or near heave, roll and pitch eigenperiods.

Low Frequency (LF) responses is defined as TLP rigid body motions at, or near surge, sway and yaw eigenperiods.

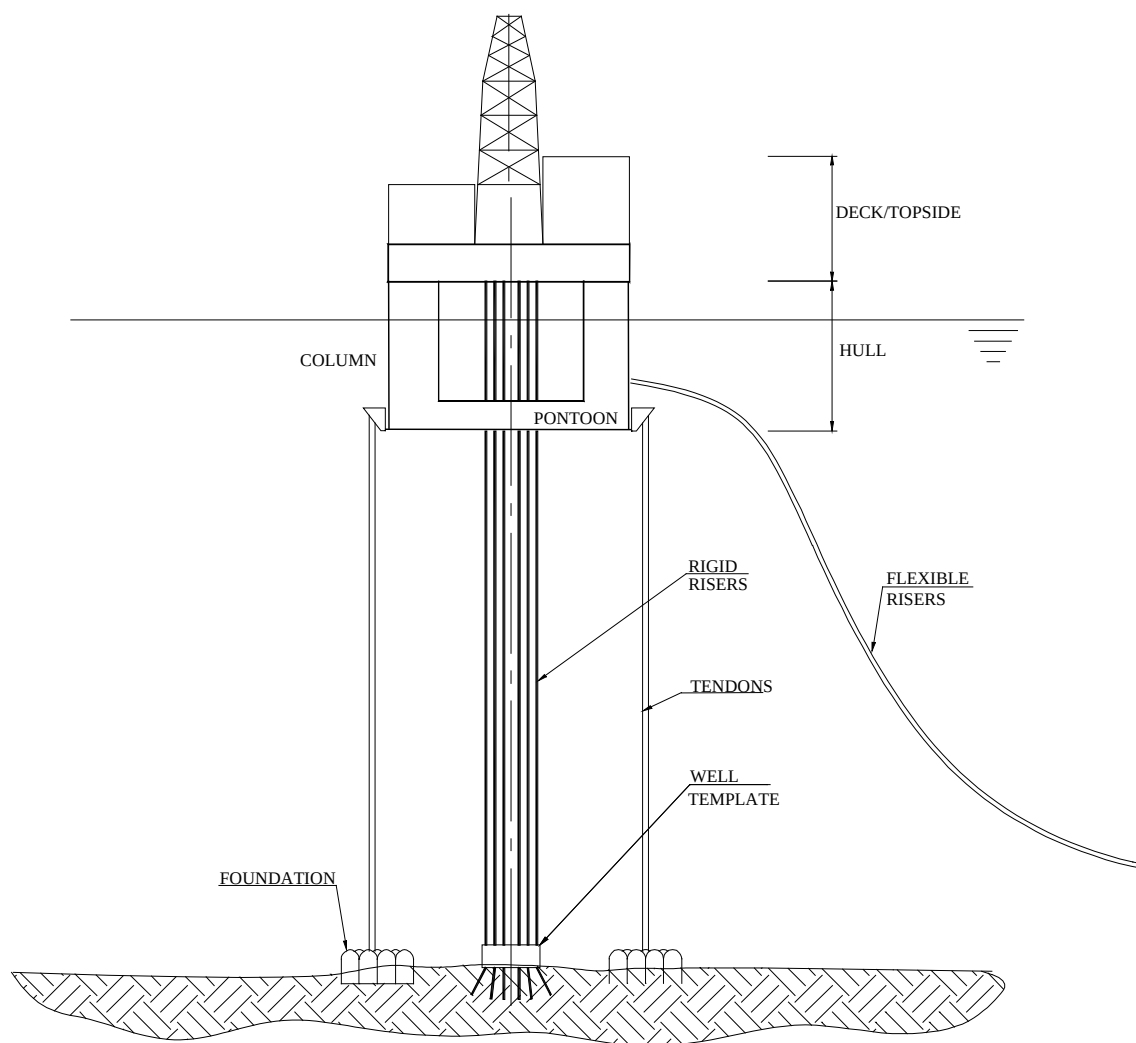


Figure N.1-1 Typical tension leg platform

N.1.3 Description of tendon system

Individual tendons are considered within this Annex as being composed of three major parts:

- Interface at the platform
- Interface at the foundation (seafloor)
- Link between platform and foundation

Tendon components at the platform interface shall adequately perform the following main functions:

- Apply, monitor and adjust a prescribed level of tension to the tendon
- Connect the tensioned tendon to the platform
- Transfer side actions and absorb bending moments or rotations of the tendon

Tendon components providing the link between the platform and the foundation consist of tendon elements (tubulars, solid rods etc.), termination at the platform interface and at the foundation interface, and intermediate connections of couplings along the length as required. The intermediate connections may take the form of mechanical couplings (threads, clamps, bolted flanges etc.), welded joints or other types of connections.

Tendon components at the foundation interface shall adequately perform the following main functions:

- Provide the structural connection between the tendon and the foundation
- Transfer side actions and absorb bending moments or rotations of the tendon

The tendon design may incorporate specialized components, such as:

- corrosion-protection system components,
- buoyancy devices,
- sensors and other types of instrumentation for monitoring the performance and condition of the tendons,
- auxiliary lines, umbilicals etc. for tendon service requirements and/or for functions not related to the tendons,
- provisions for tendons to be used as guidance structure for running other tendons or various types of equipment, and
- elastomeric elements.

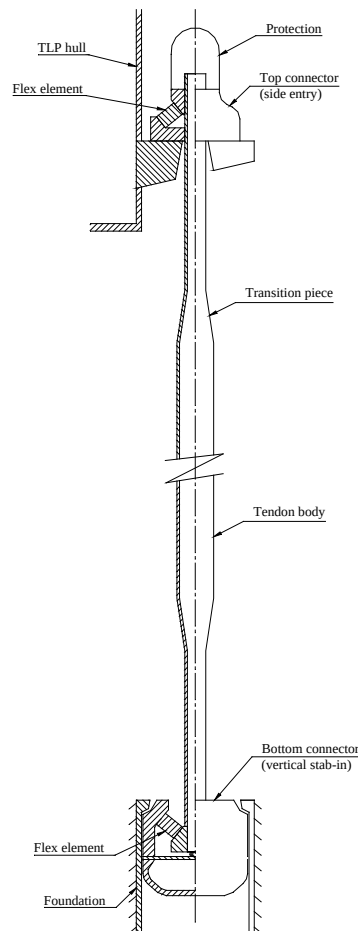


Figure N.1-2 Typical TLP tendon system

N.1.4 Non-operational phases

The structure shall be designed to resist relevant actions associated with conditions that may occur during all stages of the life-cycle of the unit. Such stages may include :

- fabrication,
- site moves,
- mating,
- sea transportation,
- installation, and,
- decommissioning.

Structural design covering marine operation, construction sequences shall be undertaken in accordance with NORSOK N-001.

Marine operations should be undertaken in accordance with the requirements stated in the VMO standard.

All marine operations shall, as far as practicable, be based upon well proven principles, techniques, systems and equipment and shall be undertaken by qualified, competent personnel possessing relevant experience.

Structural responses resulting from one temporary phase condition (e.g. a construction or transportation operation) that may effect design criteria in another phase shall be clearly documented and considered in all relevant design workings.

Fabrication

Planning of construction sequences and of the methods of construction shall be performed. Actions occurring in fabrication phases shall be assessed and, when necessary the structure and the structural support arrangement shall be evaluated for structural adequacy.

Major lifting operations shall be evaluated to ensure that deformations are within acceptable levels, and that relevant strength criteria are satisfied.

Mating

All relevant action effects incurred during mating operations shall be considered in the design process. Particular attention should be given to hydrostatic actions imposed during mating sequences.

Sea transportation

A detailed transportation assessment shall be undertaken which includes determination of the limiting environmental criteria, evaluation of intact and damage stability characteristics, motion response of the global system and the resulting, induced actions. The occurrence of slamming actions on the structure and the effects of fatigue during transport phases shall be evaluated when relevant.

In case of transportation (surface/sub surface) of tendons, this operation shall be carefully planned and analysed. Model testing shall be considered.

Satisfactory compartmentation and stability during all floating operations shall be ensured.

All aspects of the transportation, including planning and procedures, preparations, seafastening and marine operations should comply with the requirements of the Warranty Authority.

Installation

Installation procedures of foundations (e.g. piles, suction anchor or gravity based structures) shall consider relevant static and dynamic actions, including consideration of the maximum environmental conditions expected for the operations.

For novel installation activities (foundations and tendons), relevant model testing should be considered.

Tendon stand-off (pending TLP installation) phases shall be considered with respect to actions and responses.

The actions induced by the marine spread mooring involved in the operations and the forces exerted on the structures utilised in positioning the unit, such as fairleads and padeyes, shall be considered for local strength checks.

Decommissioning

Abandonment of the unit shall be planned for in the design stage.

N.2 STRUCTURAL CLASSIFICATION AND MATERIAL SELECTION

N.2.1 General

Selection of steel quality, and requirements for inspection of welds, shall be based on a systematic classification of welded joints according to the structural significance and the complexity of the joints/connections as documented in Chapter 5.

In addition to in-service operational phases, consideration shall be given to structural members and details utilised for temporary conditions, e.g. fabrication, lifting arrangements, towing and installation arrangements, etc.

N.2.2 Structural classification

The structural classification guidance given below assumes that the tendon system is demonstrated to have residual strength and that the TLP structural system satisfies the requirements of the ALS condition with failure of the tendon (or a connection in the tendon) as the defined damage. (See also NORSOK N-004, Table 5-1). If this is not the case then design classes, DC3 and DC4 stated below should be substituted by design classes DC1 and DC2 and the inspection categories respectively upgraded in accordance with the requirements of NORSOK N-004, Tables 5-3 and 5-4 as applicable.

Examples of typical design classes applicable to the hull and deck structures of a TLP are given in Annex M.

Examples of typical design classes applicable to the pile foundation structures are given in Annex K.

Examples of considerations with respect to structural classification of tendons, tendon interfaces are given below. These examples provide minimum requirements and are not intended to restrict the designer in applying more stringent requirements should such requirements be desirable.

Typical locations - Design class : DC3

Locations in way of complex structural connections should be classified as DC3, such connections may occur at :

- tendon interfaces with the foundation and the TLP hull, and,
- complex tendon / tendon connections.

DC3 areas may be limited to local, highly stressed areas if the stress gradient at such connections is large.

Typical locations - Design class : DC4

Except as provided for in the description for DC3 structural categorisation, the following listed connections may appropriately be classified as being DC4 :

- simple tendon / tendon connections,
- interface arrangements outside locations of complex connections including general stiffened plate fields (e.g. at hull interface)

Consideration of the economic consequence of failure may however indicate the utilisation of a higher design class than the minimum class as given above.

Typical locations - Design class : DC5

DC5 locations are not normally relevant for tendons or tendon interfaces.

N.2.3 Material selection

Material specifications shall be established for all structural materials utilised in a TLP. Such materials shall be suitable for their intended purpose and have adequate properties in all relevant design conditions. Material selection shall be undertaken in accordance with the principles given in NORSOK M-DP-001.

When considering criteria appropriate to material grade selection, adequate consideration shall be given to all relevant phases in the life cycle of the unit. In this connection there may be conditions and criteria, other than those from the in-service, operational phase, that provide the design requirements in respect to the selection of material. (Such criteria may, for example, be design temperature and/or stress levels during marine operations.)

Selection of steel quality for structural components shall normally be based on the most stringent Design Class of the joints involving the component.

Through-thickness stresses and low temperature toughness requirements shall be assessed.

The evaluation of structural resistance shall include relevant account of variations in material properties for the selected material grade (e.g. Variation in yield stress as a function of thickness of the base material)

N.2.4 Inspection categories

Welding, and the extent of non-destructive examination during fabrication, shall be in accordance with the requirements stipulated for the appropriate 'inspection category' as defined in NORSOK, M-101.

Determination of inspection category should be in accordance with the criteria given in Table 5-5.

The lower the extent of NDT selected, the more important is the representativeness of the NDT selected and performed. The designer should therefore exercise good engineering judgement in indicating mandatory locations for NDT, where variation of utility along welds is significant.

N.2.5 Guidance to minimum requirements

Figures N.2-1 and N.2-2 illustrate minimum requirements to selection of Design Class, and Inspection Category for typical TLP unit, structural configurations.

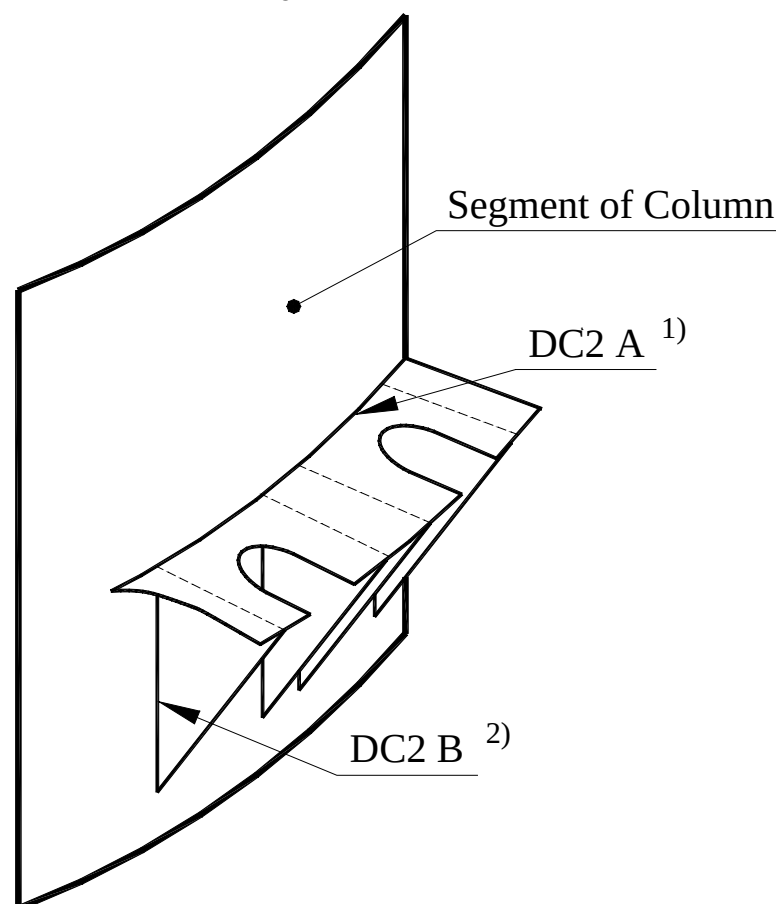


Figure N.2-1 Examples of typical hull porch design classes and inspection categories

1. *Inspection Category A is selected in accordance with Table 5.4 assuming high fatigue utilisation and principal stresses in the transverse direction.*
2. *Inspection Category B is selected in accordance with Table 5.4 assuming high fatigue and principal stresses longitudinal to the weld.*

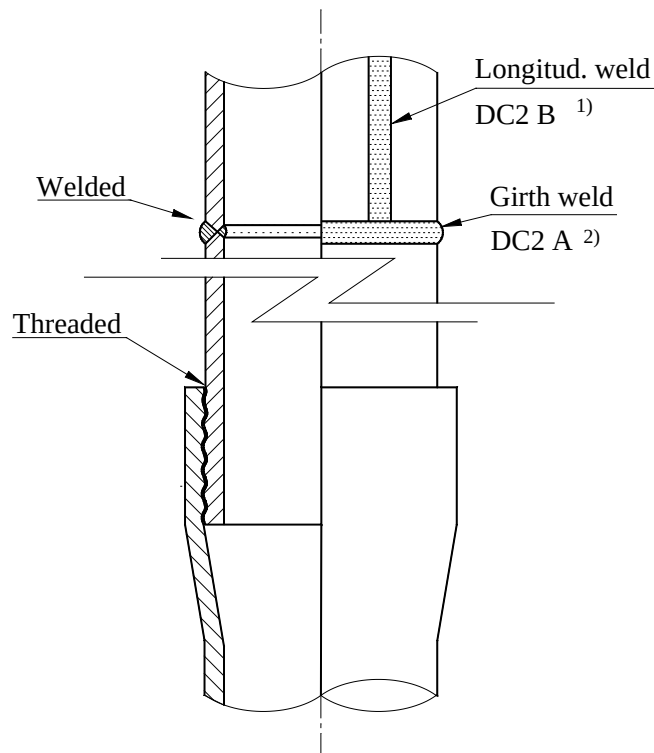


Figure N.2-2 Examples of design classes and inspection categories for typical tendon connections

1. *Inspection Category B is selected in accordance with Table 5.4 assuming high fatigue utilisation and principal stresses in the longitudinal direction.*
2. *Inspection Category A is selected in accordance with Table 5.4 assuming high fatigue and principal stresses transverse to the weld. Stricter acceptance criteria as per footnote 2 in Table 5.4 will apply.*

N.3 DESIGN CRITERIA

N.3.1 General

The following basic design criteria shall be complied with for the TLP design:

- The TLP is to be able to sustain all actions liable to occur during all relevant temporary and operating design conditions for all applicable limit states.
- Direct wave actions on the deck structure should not occur in the ultimate limit-state (ULS). Direct wave actions on the deck structure may be accepted in the ALS condition provided that such actions are adequately included in the design.

Operating tolerances shall be specified and shall be achievable in practice. Normally, the most unfavourable operating tolerances shall be included in the design. Active operation shall not to be dependent on high reliability of operating personnel in an emergency situation.

Note: Active operation of the following may be considered in an emergency situation, as applicable:

- Ballast distribution
- Weight distribution
- Tendon tension
- Riser tension

N.3.2 Design principles, tendons

Essential components of the tendon system shall be designed by the principle that, as far as practicable, they are to be capable of being inspected, maintained, repaired and/or replaced.

Tendon mechanical components shall, as far as practicable, be designed "fail to safe". Consideration is to be given in the design to possible early detection of failure for essential components which cannot be designed according to this principle.

Certain vital tendon components may, due to their specialised and unproven function, require extensive engineering and prototype testing to determine:

- Confirmation of anticipated design performance
- Fatigue characteristics
- Fracture characteristics
- Corrosion characteristics
- Mechanical characteristics

The tendon system and the securing/supporting arrangements shall be designed in such a manner that a possible failure of one tendon is not to cause progressive tendon failure or excessive damage to the securing/supporting arrangement at the platform or at the foundation.

A fracture control strategy should be adopted to ensure consistency of design, fabrication and in service monitoring assumptions. The object of such a strategy is to ensure that the largest undetected flaw from fabrication of the tendons will not grow to a size that could induce failure within the design life of the tendon, or within the planned in-service inspection interval, within a reasonable level of reliability. Elements of this strategy include:

- Design fatigue life
- Fracture toughness
- Reliability of inspection during fabrication
- In-service inspection intervals and methods

Fracture mechanics should be used to define allowable flaw sizes, estimate crack growth rates and thus help define inspection intervals and monitoring strategies.

All materials liable to corrode shall be protected against corrosion. Special attention should be given to:

- Local complex geometries
- Areas that are difficult to inspect/repair
- Consequences of corrosion damage

- Possibilities for electrolytic corrosion

All sliding surfaces shall be designed with sufficient additional thickness against wear. Special attention should be given to the following:

- Cross-load bearings
- Seals
- Balljoints

Satisfactory considerations shall be given to settlement/subsidence which may be a significant factor in determining tendon-tension adjustment requirements.

N.4 ACTIONS

N.4.1 General

Characteristic actions are to be used as reference actions. Design actions are, in general, defined in NORSOK N-003. Guidance concerning action categories relevant for TLP designs are given in the following.

N.4.2 Different actions

All relevant actions that may influence the safety of the structure or its parts from commencement of construction to permanent decommissioning should be considered in design. The different actions are defined in NORSOK N-003. For the deck and hull of the TLP, the actions are similar to those also described in Annex M. Additional actions on the hull are those from ringing and springing.

The wave actions on the tendons can be described as recommended in NORSOK N-003 for slender structures with significant motions.

If of importance, the disturbance from hull and risers on the wave kinematics at the tendon locations should be accounted for.

The earthquake actions at the foundation of the tendons are described according to NORSOK N-003.

The following actions should be considered:

- Permanent Actions
- Variable Actions
- Deformation Actions
- Accidental Actions
- Environmental Actions

The environmental actions are to be summarized as:

- Wind Actions
 - Mean wind
 - Dynamic wind
 - Local wind
- Wave and current Actions
 - Actions on slender members
 - Actions induced by TLP motions
 - Slamming and shock pressure
 - Wave diffraction and radiation
 - Mean drift forces
 - Higher order non-linear wave actions (slowly varying, ringing and springing)
 - Wave enhancement
 - Vortex shedding effects
- Marine growth
- Snow and ice accumulation
- Direct ice action (icebergs and icefloes)
- Earthquake
- Temperature
- Tidal effects

N.5 GLOBAL PERFORMANCE

N.5.1 General

The selected methods of response analysis are dependent on the design conditions, dynamic characteristics, non-linearities in action and response and the required accuracy in the actual design phase. For a detailed discussion of the different applicable methods for global analysis of tension leg platforms, reference is made to API RP 2T, /5/.

The selected methods of analysis and models employed in the analysis shall include relevant non-linearities and motion-coupling effects. The approximations, simplifications and/or assumptions made in the analysis shall be justified, and their possible effects shall be quantified e.g. by means of simplified parametric studies. During the design process, the methods for analytical or numerical prediction of important system response shall be verified (calibrated) by appropriate model tests.

Model tests may also be used to determine specific responses for which numerical or analytical procedures are not yet developed and recognized.

Relevant motions shall be determined, by relevant analysis techniques, for those applicable design conditions specified in NORSOK N-003. The basic assumptions and limitations associated with the different methods of analysis of global performance shall be duly considered prior to the selection of the methods.

The TLP should be analysed by methods as applicable to column-stabilised units (ref. Annex M) when the unit is free floating.

The method of platform-motion analysis as outlined in this Annex is one approximate method which may be applied. The designer is encouraged also to consider and apply other methods in order to discover the effects of possible inaccuracies etc. in the different methods.

N.5.2 Frequency domain analysis

Frequency domain HF, WF and LF analyses techniques may be applied for a TLP. Regarding action effects due to mean wind, current and mean wave drift, see NORSOK N-003.

For typical TLP geometries and tendon arrangements, the analysis of the total dynamic action effects may be carried out as:

- A high-frequency (HF) analysis of springing
- A wave-frequency (WF) analysis in all six degrees of freedom
- A low-frequency (LF) analysis in surge, sway and yaw

The following assumptions are inherent in adopting such an independent analysis approach:

- The natural frequencies in heave, pitch and roll are included in the wave-frequency analysis
- The natural frequencies in surge, sway and yaw are included in the low-frequency analysis
- The high and low natural frequencies are sufficient separate to allow independent dynamic analysis to be carried out
- The low-frequency excitation forces have negligible effect on the wave-frequency motions
- The low-frequency excitation forces have a negligible dynamic effect in heave, pitch and roll
- Tendon lateral dynamics are unimportant for platform surge/sway motions

Typical parameters to be considered for global performance analyses are different platform draft, tidal effects, storm surges, set down, settlement, subsidence, mispositioning and tolerances. Possible variations in vertical centre of gravity shall also be analysed (especially if ringing responses are important).

N.5.2.1 High frequency analyses

Frequency domain springing analyses shall be performed to evaluate tendon and TLP susceptibility to springing responses.

Recognised analytical methods exist for determination of springing responses in tendons. These methods include calculation of Quadratic Transfer Functions for axial tendon (due to sum frequency actions on the hull) stresses which is the basis for determination of tendon fatigue due to springing.

Damping level applied in the springing response analyses shall be duly considered and documented.

N.5.2.2 Wave-frequency analyses

A wave-frequency dynamic analysis may normally be carried out by using linear wave theory in order to determine first-order platform motions and tendon response.

First order wave action analyses shall also serve as basis for structural response analyses. Finite wave action effects shall be evaluated and taken into account.

In linear theory, the response in regular waves (transfer functions) is combined with a wave spectrum to predict the response in irregular seas.

The effect of low-frequency set-down variations on the WF analysis is to be investigated by analysing at least two representative mean offset positions determined from the low-frequency analysis.

Wave approach headings shall be selected with basis in global configuration (e.g. number of columns).

N.5.2.3 Low-frequency analyses

A low-frequency dynamic analysis could be performed to determine the slow drift effects at early design stage due to fluctuating wind and second order wave actions.

Appropriate methods of analysis shall be used with selection of realistic damping levels. Damping coefficients for low-frequency motion analyses are important as the low-frequency motion may be dominated by resonant response.

N.5.3 Time domain analyses

For global motion response analyses, a time domain approach will be beneficial. In this type of analyses it is possible to include all environmental action effects and typical non-linear effects such as:

- Hull drag forces (including relative velocities)
- Finite wave amplitude effects
- Non-linear restoring (tendons, risers)

Highly non-linear effects such as ringing may also require a time domain analysis approach. Analytical methods exist for estimation of ringing responses. These methods can be used for the early design stage, but shall be correlated against model tests for the final design. Ringing and springing responses of hull and deck may however be analysed within the frequency domain with basis in model test results, or equivalent analytical results.

For deep waters a fully coupled time domain analysis of tendons, risers and platform may be required.

A relevant wave spectrum shall be used to generate random time series when simulating irregular wave elevations and kinematics.

Simulation length shall be long enough to obtain sufficient number of LF maxima (surge, sway, yaw).

Statistical convergence shall be checked by performing sensitivity analyses where parameters as input seed, simulation length, time step, solution technique etc. are varied.

Determination of extreme responses from time domain analyses shall be performed according to recognised principles.

Depending on selected TLP installation method, time domain analyses will probably be required to simulate the situation when the TLP is transferred from a free floating mode to the vertical restrained mode. Model testing shall also be considered in this context.

N.5.4 Model testing

Model testing will usually be required for final check of TLP designs. The main reason for model testing is to check that analytical results correlate with model tests.

The most important parameters to evaluate are:

- Air-gap
- First order motions
- Total offset
- Set-down
- WF motions versus LF motions
- Accelerations
- Ringing
- Springing

The model scale applied in testing shall be appropriate such that reliable results can be expected. A sufficient number of seastates need to be calibrated covering all limit states.

Wave headings and other variable parameters (water levels, vertical centre of gravity, etc.) need to be varied and tested as required.

If HF responses (ringing and springing) shows to be governing for tendon extreme and fatigue design respectively, the amount of testing may have to be increased to obtain confidence in results.

N.5.5 Action effects in the tendons

Action effects in the tendons comprise of steady-state and dynamic components.

The steady-state actions may be determined from the equilibrium condition of the platform, tendon and risers.

Tendon action effects arise from platform motions, any ground motions and direct hydrodynamic actions on the tendon.

Dynamic analysis of tendon response shall take into account the possibility of platform pitch, roll and heave excitation (springing and ringing effects).

Linearized dynamic analysis does not include some of the secondary wave effects, and may not model accurately extreme wave responses. A check of linear-analysis results using non-linear methods may be necessary. Model testing may also be used to confirm analytical results. Care shall be exercised in interpreting model-test results for resonant responses, particularly for actions due to platform roll, pitch and heave, since damping may not be accurately modelled.

Lift and overturning moment generated on the TLP hull by wind actions shall be included in the tendon response calculations.

Susceptibility to vortex induced vibrations shall be evaluated in operational and non-operational phases.

Interference (tendon/riser, tendon/tendon, tendon/hull, tendon/foundation) shall be evaluated for non-operational as well as the operational phase.

N.6 ULTIMATE LIMIT STATE (ULS)

N.6.1 General

General considerations in respect to methods of analysis are given in NORSOK N-003.

The TLP hull shall be designed for the loading conditions that will produce the most severe effects on the structure. A dynamic analysis shall be performed to derive at characteristic largest stresses in the structure. Analytical models shall adequately describe the relevant properties of actions, stiffness and displacement, and shall account for the local and system effects of, time dependency, damping and inertia.

It is normally not practical, in design analysis of TLP's, to include all relevant actions (both global and local) in a single model. Generally, a single model would not contain sufficient detail to establish local responses to the required accuracy, or to include consideration of all relevant actions and combinations of actions. It is often the case that it is more practical, and efficient, to analyse different action effects utilising a number of appropriate models and superimpose the response from one model with the responses from another model in order to assess the total utilisation of the structure. For preliminary design, simplified models are recommended to be utilised in order to more efficiently establish design responses and to achieve a simple overview of how the structure responds to the designing actions. For final design, a complete three dimensional model of the platform is required.

N.6.2 Hull

The following analysis procedure to obtain characteristic platform-hull response shall be applied:

a) *Steady-state analysis of the initial position.*

In this analysis, all vertical actions are applied (weights, live loads, buoyancy etc.) and equilibrium is achieved taking into account pretension in tendons and risers.

b) *Steady-state offset*

In this analysis the lateral steady-state wind, wave-drift and current actions are applied to the TLP resulting in a static offset position with a given set-down.

c) *Design wave analysis*

To satisfy the need for simultaneity of the responses, a design wave approach, see NORSOK N-003, may normally be used for maximum stress analysis.

The merits of the stochastic approach are retained by using the extreme stochastic values of some characteristic parameters in the selection of the design wave and is applied to the platform in its offset position. The results are superimposed on the steady-state solution to obtain maximum stresses.

d) *Spectral analysis*

Assuming the same offset position as described in b) and with a relevant storm spectrum, an analysis is carried out using 'n' wave frequencies from 'm' directions. Traditional spectral analysis methods should be used to compute the relevant response spectra and their statistics.

For a TLP hull, the following characteristic global sectional actions due to wave forces shall be considered as a minimum, see also Annex M:

- Split forces (transverse, longitudinal or oblique sea for odd columned TLP's)
- Torsional moment about a transverse and longitudinal, horizontal axis (in diagonal or near-diagonal)
- Longitudinal opposed forces between parallel pontoons (in diagonal or near-diagonal seas)
- Longitudinal, transverse and vertical accelerations of deck masses

It is recommended that a full stochastic wave action analysis is used as basis for the final design.

Local load effects (e.g. maximum direct environmental action of an individual member, wave slamming loads, external hydrostatic pressure, ballast distribution, internal tank pressures etc.) shall be considered. Additional actions from e.g. high-frequency ringing accelerations shall be taken into account.

N.6.2.1 Structural analysis

For global structural analysis, a complete three-dimensional structural model of the platform is required.

Note: *Linear elastic space-frame analysis may be utilised if the torsional moments, for example as resulting from diagonal seas, are transferred mainly through a stiff bracing arrangement. Otherwise, finite-element analysis is required, see also Annex M..*

Additional detailed finite-element analyses may be required for complex joints and other complicated structural parts to determine the local stress distribution more accurately and/or to verify the results of a space-frame analysis, see also Annex M.

Where relevant local stress concentrations shall be determined by detailed finite-element analysis or by physical models. For standard details, however, recognised formulas will be accepted.

Supplementary manual calculations for members subjected to local actions may be required where appropriate.

If both static and dynamic action contributions are included in one analysis, the results shall be such that the contributions from both shall be individually identifiable.

Local environmental action effects, such as wave slamming and possible wave- or wind-induced vortex shedding, are to be considered as appropriate.

N.6.2.2 Structural design

Special attention shall be given to the structural design of the tendon supporting structures to ensure a smooth transfer and redistribution of the tendon concentrated actions through the hull structure without causing undue stress concentrations.

The internal structure in columns in way of bracings should to be designed stronger than the axial strength of the bracing itself.

Special consideration shall be given to the pontoon strength in way of intersections with columns, accounting for possible reduction in strength due to cut-outs and stress concentrations.

N.6.3 Deck

N.6.3.1 General

Structural analysis design of deck structure shall follow the principles as outlined in NORSOK N-004, Annex M. Additional actions (e.g. global accelerations) from high-frequency ringing and springing shall be taken into account when relevant.

N.6.3.2 Air gap

Requirements and guidance to air gap analyses for a TLP unit are given in NORSOK N-003.

In the ULS condition, positive air gap should be ensured. However, wave impact may be permitted to occur on any part of the structure provided that it can be demonstrated that such actions are adequately accounted for in the design and that safety to personnel is not significantly impaired.

Analysis undertaken to document air gap should be calibrated against relevant model test results. Such analysis shall include relevant account of:

- wave/structure interaction effects,
- wave asymmetry effects,
- global rigid body motions (including dynamic effects),
- effects of interacting systems (e.g. riser systems), and,
- maximum/minimum draughts (setdown, tidal surge, subsidence, settlement effects).

Column 'run-up' action effects shall be accounted for in the design of the structural arrangement in way of the column/deckbox connection. These 'run-up' actions should be treated as an environmental action component, however, they need not normally be considered as occurring simultaneously with other environmental responses.

Evaluation of air gap adequacy shall include consideration of all affected structural items including lifeboat platforms, riser balconies, overhanging deck modulus etc.

N.6.4 Tendons

N.6.4.1 *Extreme tendon tensions*

As a minimum the following tension components shall be taken into account:

- Pretension (static tension at MSL)
- Tide (tidal effects)
- Storm surge (positive and negative values)
- Tendon weight (submerged weight)
- Overturning (due to current, mean wind/drift load)
- Setdown (due to current, mean wind/drift load)
- WF tension (wave frequency component)
- LF tension (wind gust and slowly varying drift)
- Ringing (HF response)

Additional components to be considered are:

- Margins for fabrication, installation and tension reading tolerances.
- Operational requirements (e.g. operational flexibility of ballasting operations)
- Allowance for foundation mispositioning
- Field subsidence
- Foundation settlement and uplift

Bending stresses along the tendon shall be analysed and taken into account in design. For the constraint mode the bending stresses in tendon will usually be low. In case of surface, or subsurface tow (non-operational phase) the bending stresses shall be carefully analysed and taken into account in design. For nearly buoyant tendons the combination of environmental action (axial & bending) and high hydrostatic water pressure may be a governing combination.

Limiting combinations of tendon tension and rotations (flex elements) need to be established.

For specific tendon components such as couplings, flex elements, top and bottom connections etc. the stress distribution shall be determined by appropriate finite-element analysis.

If tendon tension loss is permitted, tendon dynamic analyses shall be conducted to evaluate its effect on the complete tendon system. Alternatively model tests may be performed. The reasoning behind this is that loss of tension could result in detrimental effects from tendon buckling and/or damage to flex elements.

N.6.4.2 *Structural design*

The structural design of tendons shall be carried out according to NORSOK N-001 and N-003 with the additional considerations given in this subsection.

When deriving maximum stresses in the tendons relevant stress components shall be superimposed on the stresses due to maximum tendon tension, minimum tendon tension or maximum tendon angle, as relevant.

Such additional stress components may be:

- Tendon-bending stresses due to lateral actions and motions of tendon
- Tendon-bending stresses due to flexelement rotational stiffness
- Thermal stresses in the tendon due to temperature differences over the cross sections
- Hoop stresses due to hydrostatic pressure

N.6.5 Foundations

Geotechnical field investigations and careful data interpretation shall form the basis for geotechnical design parameters.

Relevant combinations of tendon tensions and angles shall be analysed for the foundation design.

For gravity foundations the pretension shall be compensated by submerged weight of the foundation, whereas the varying actions may be resisted by for example suction and friction.

N.7 FATIGUE LIMIT STATE (FLS)

N.7.1 General

Structural parts where fatigue may be a critical mode of failure shall be investigated with respect to fatigue. All significant actions contributing to fatigue damage (non-operational and operational) design conditions shall be taken into account. For a TLP, the effects of springing and ringing resonant responses shall be considered for the fatigue limit state.

Fatigue design may be carried out by methods based on fatigue tests and cumulative damage analysis, methods based on fracture mechanics, or a combination of these.

General requirements to fatigue design are given in Chapter 8 and Annex C.

Careful design of details as well as stringent quality requirements to fabrication are essential in achieving acceptable fatigue strength. It is to be ensured that the design assumptions made concerning these parameters are achievable in practice.

The results of fatigue analyses shall be fully considered when the in-service inspection plans are developed for the platform.

N.7.2 Hull

Fatigue design of hull structure shall be performed in accordance to principles given in Annex M.

N.7.3 Deck

Fatigue design of deckstructure shall be performed in accordance to principles given in Annex M.

N.7.4 Tendons

All parts of the tendon system shall be evaluated for fatigue.

First order wave actions (direct/indirect) will usually be governing, however also fatigue due to springing shall be carefully considered. HF and WF tendon responses shall be combined realistically.

In case of wet transportation (surface/subsurface) to field these fatigue contributions shall be accounted for in design.

Vortex Shedding shall be considered and taken into account. This applies to operation and non-operational (e.g. tendon stand-off) phases.

Series effects (welds, couplings) shall be evaluated.

When fracture-mechanics methods are employed, realistic estimates of strains combined with maximum defect sizes likely to be missed with the applicable NDE methods shall be used

N.7.5 Foundation

Tendon responses (tension and angle) will be the main contributors for fatigue design of foundations. Local stresses shall be determined by use of finite element analyses.

N.8 SERVICEABILITY LIMIT STATE (SLS)

N.8.1 General

Considerations shall be given to the effect of deflections where relevant.

The stiffness of the structure and structural components shall be sufficient to prevent serious vibrations and to ensure safe operation of the platform.

Allowance for wear shall be considered in areas exposed to abrasion (e.g where cross load bearings may move relative to the TLP column).

N.9 ACCIDENTIAL LIMIT STATE (ALS)

N.9.1 General

General guidance and requirements concerning accidental events are given in Chapter 9 and Annex A.

Units shall be designed to be damage tolerant, i.e. credible accidental damages, or events, should not cause loss of global structural integrity. The capability of the structure to redistribute actions should be considered when designing the structure.

In the design phase, attention shall be given to layout and arrangements of facilities and equipment in order to minimise the adverse effects of accidental events.

Satisfactory protection against accidental damage may be obtained by a combination of the following principles :

- reduction of the probability of damage to an acceptable level, and,
- reduction of the consequences of damage to an acceptable level.

Structural design in respect to the Accidental damage Limit State (ALS) shall involve a two-stage procedure considering :

- resistance of the structure to a relevant accidental event, and,
- capacity of the structure after an accidental event.

Global structural integrity shall be maintained both during and after an accidental event. Actions occurring at the time of a design accidental event and thereafter shall not cause complete structural collapse.

Requirements to compartmentation and stability in the damage condition are given in N10. When the upper hull (deckbox) structure becomes buoyant in satisfying requirements to damage stability, /2/, consideration shall be given to the structural response resulting from such actions.

N.9.2 Hull and deck

The most relevant accidental events for hull and deck design are:

- dropped objects
- fire
- explosion
- collision
- unintended flooding

Compartmentation is a key issue for TLP's due to the fine balance between weight, buoyancy and pretensions. See N10.2.

N.9.3 Tendons

The most relevant accidental events for the tendons are:

- missing tendon
- tendon flooding
- dropped objects
- hull compartment(s) flooding

Missing tendon requires analysis with 100 year environment to satisfy the ALS. The same applies to tendon flooding, if relevant.

For accidental events leading to tendon failure the possible detrimental effect of the release of the elastic energy stored in the tendon may have on the surrounding structure shall be considered.

Dropped object may cause damage to the tendons and in particular the top and bottom connectors may be exposed. Shielding may be required installed.

Flooding hull compartments and the effects on design shall be analysed thoroughly.

N.9.4 Foundations

Accidental events to be considered for the foundations shall as a minimum be those listed for tendons.

N.10 COMPARTMENTATION & STABILITY

N.10.1 General

The TLP shall have draught marks pertinent to all phases of operation. The draught marks shall be located forward and aft on the platform and at starboard and port sides. The draught marks shall be clearly visible for operating personnel, unless instruments for continuous draught sensing and reading are provided.

When the construction of the platform is as close to completion as practical, an inclining test shall be undertaken in order to establish the position of the centre of gravity and the light weight.

The information obtained from the inclining test is to be continuously updated and adjustments shall be made to account for any items taken on or off the TLP after such tests.

Loading conditions during operating or temporary phases of the TLP shall have loads and centre-of-gravity location within the envelope of maximum/minimum allowable values. Information concerning these requirements is to be specified in the Operating Manual of the TLP.

Detailed provisions as stated in IMO MODU code, /4/ shall be satisfied.

N.10.2 Compartmentation and watertight integrity

Temporary conditions

For temporary phases where the tendons are not installed, the platform may be considered as a column-stabilised unit for which procedures and criteria concerning hydrostatic stability should be taken according to Annex M.

For temporary phases where tendons are partly engaged, adequate stability shall be documented. The stability in this condition may be provided by hydrostatic stability, by the tensioned tendons or by a combination.

Operating conditions

The tension in the tendons is to be sufficient to ensure the stability of the platform in the operating phase, both for the intact and damaged conditions.

Monitoring of weight, ballast and pretension shall be performed.

The following minimum accidental flooding criteria is normally to be assumed for the ALS condition:

- a) Any one compartment adjacent to the sea is to be assumed flooded, regardless of cause.
- b) Any one compartment containing sea water piping systems of other sources containing liquids, due to failure of such arrangements, is to be assumed flooded unless it can be adequately documented that such requirement is unwarranted.

Watertight integrity

Watertight closing appliances are required for those external openings up to the water levels defined by:

- The water level for an angle of heel equal to the first intercept in the intact or damage condition, whichever is greater (free-floating conditions)
- The water level corresponding to the required air-gap for deck clearance in the ULS condition

Where pipe-runs may lead to critical flooding scenarios, full account is to be taken of this possibility in the design of these pipe-runs and the size of the internal spaces they run to or from.

The number of openings in watertight bulkheads and decks are to be kept to a minimum compatible with the design and proper operation of the TLP. Where penetration of watertight decks and bulkheads are necessary for access, piping, ventilation, electrical cables etc., arrangements are to be made to maintain the watertight integrity.

Where valves are provided at watertight boundaries to provide watertight integrity, these valves are to be capable of being operated from a normally manned space. Valve position indicators are to be provided at a remote-control situation.

Watertight doors and hatches are to be remotely controlled from a central safe position and are also to be operable locally from each side of the bulkhead or deck. Indicators are to be provided at the remote-control position to indicate whether the doors are open or closed.

Where the tendon elements and the tendon-column annulus are designed as non-flooded members in service, such compartments are to be considered as TLP buoyancy compartments with respect to watertight integrity. Special consideration of closing devices (seals) for the tendon/column duct annulus is necessary to fulfil requirements as to "watertight closure".

N.11 DOCUMENTATION REQUIREMENTS FOR THE ‘DESIGN BASIS’ & ‘DESIGN BRIEF’

N.11.1 General

Adequate planning shall be undertaken in the initial stages of the design process in order to obtain a workable and economic structural solution to perform the desired function. As an integral part of this planning, documentation shall be produced identifying design criteria and describing procedures to be adopted in the structural design of the unit.

Applicable codes, standards and regulations shall be identified at the commencement of the design.

When the design has been finalised, a summary document containing all relevant data from the design and fabrication phase shall be produced.

Design documentation (see below) shall, as far as practicable, be concise, non-voluminous, and, should include all relevant information for all relevant phases of the lifetime of the unit.

General requirements to documentation relevant for structural design are given in NORSOK, N-001, Section 5.

N.11.2 Design basis

A Design Basis Document shall be created in the initial stages of the design process to document the basis criteria to be applied in the structural design of the TLP.

A summary of those items normally to be included in the Design Basis document is included below.

Unit description and main dimensions

A summary description of the unit, including :

- general description of the unit (including main dimensions and draughts),
- main drawings including :
 - main structure drawings (This information may not be available in the initial stage of the design),
 - general arrangement drawings,
 - plan of structural categorisation,
 - load plan(s), showing description of deck uniform (laydown) actions,
 - capacity (tank) plan,
- service life of unit,
- tendon system description.
- foundation system.

Rules, regulations and codes

A list of all relevant, applicable, Rules, Regulations and codes (including revisions and dates), to be utilised in the design process.

Environmental design criteria

Environmental design criteria (including all relevant parameters) for all relevant conditions, including :

- wind, wave, current, snow and ice description for 10^{-1} , 10^{-2} and 10^{-4} annual probability events
- water levels (tides, surges, subsidence)
- design temperatures

Stability and compartmentation

Stability and compartmentation design criteria for all relevant conditions including :

- external and internal watertight integrity plan
- lightweight breakdown report
- design loadcase(s) including global mass distribution
- damage condition waterlines.(This information may not be available in the initial stage of the design.)

Temporary phases

Design criteria for all relevant temporary phase conditions including, as relevant:

- limiting permanent, variable, environmental and deformation action criteria
- procedures associated with construction, (including major lifting operations)
- essential design parameters associated with temporary phases (e.g. for mating loadcases, mating weld-up sequences, crushing tube stiffness', for transit phases, transit speed etc.)
- relevant ALS criteria
- tendon installation/replacement
- foundation installation

Operational design criteria

Design criteria for all relevant operational phase conditions including :

- limiting permanent, variable, environmental and deformation action criteria
- designing accidental event criteria (e.g. collision criteria)
- tank loading criteria (all tanks) including a description of system, with :
 - loading arrangements
 - height of air pipes
 - loading dynamics
 - densities
- tendon criteria
- tendon monitoring philosophy
- foundation criteria
- tendon interface criteria (functionality, stiffness etc.)

Air gap

Relevant basis information necessary for the assessment of air gap sufficiency, including:

- a description of the requirements to be applied in the ULS and ALS conditions
- basis model test report (This information may not be available in the initial stage of the design)

In-service inspection criteria

A description of the in-service inspection criteria and general philosophy (as relevant for evaluating fatigue allowable cumulative damage ratios).

Miscellaneous

A general description of other essential design information, including :

- description of corrosion allowances, where applicable

N.11.3 Design brief

A Design Brief Document shall be created in the initial stages of the design process. The purpose of the Design Brief shall be to document the intended procedures to be adopted in the structural design of the TLP. All applicable limit states for all relevant temporary and operational design conditions shall be considered in the Design Brief.

A summary of those items normally to be included in the Design Brief Document is included below.

Analysis models

A general description of models to be utilised, including description of :

- global analysis model(s)
- local analysis model(s)
- loadcases to be analysed

Analysis procedures

A general description of analytical procedures to be utilised including description of procedures to be adopted in respect to :

- the evaluation of temporary conditions

- the consideration of accidental events
- the evaluation of fatigue actions
- air gap evaluation, (including locations to be considered, damping, inclusion of asymmetry factors, disturbed (radiated) wave considerations, combined motion response)
- the establishment of dynamic responses (including methodology, factors, and relevant parameters)
- the inclusion of 'built-in' stresses
- the consideration of local responses (e.g. those resulting from tendon and riser actions, ballast distribution etc.)
- the consideration of structural redundancy

Structural evaluation

A general description of the evaluation process, including :

- description of procedures to be utilised for considering global and local responses
- description of fatigue evaluation procedures (including use of design fatigue factors, SN-curves, basis for stress concentration factors (SCF's), etc.)
- description of procedures to be utilised for code checking

Air gap evaluation

A general description of the air gap evaluation procedure, including :

- description of procedures to be utilised for considering air gap sufficiency

N.12 REFERENCES

- /1/ Regulations of 20 Dec. 1991, No.878, concerning stability, watertight subdivision and watertight/weathertight closing means on mobile offshore units, Norwegian Maritime Directorate.
- /2/ ISO 19900, Petroleum and Natural Gas Industries-Offshore Structures: General Requirements, International Standard,.
- /3/ Code for the Construction and Equipment of Mobile Offshore Drilling Units, 1989, (1989 MODU CODE), International Maritime Organisation, London 1990.
- /4/ API RP 2T Recommended Practice for Planning, Designing, and Constructing Tension Leg Platforms, 3. edition August 2012.

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