IACS Common Structural Rules for Double Hull Oil Tankers, January 2006

Background Document

SECTION 10 – BUCKLING AND ULTIMATE STRENGTH

NOTE:

- This TB is published to improve the transparency of CSRs and increase the understanding of CSRs in the industry.
- The content of the TB is not to be considered as requirements.
- This TB cannot be used to avoid any requirements in CSRs, and in cases where this TB deviates from the Rules, the Rules have precedence.
- This TB provides the background for the first version (January 2006) of the CSRs, and is not subject to maintenance.

IACS

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1 GENERAL

1.1 Strength Criteria

1.1.1 Scope

1.1.1.a The buckling and ultimate strength requirements in the Rules are organised in three different categories. The categories are:

(a) Stiffness and Proportions (*Section 10/2 of the Rules*). Non-stress based requirements. The requirements are typically defined as maximum allowable slenderness ratios or minimum inertia requirements.

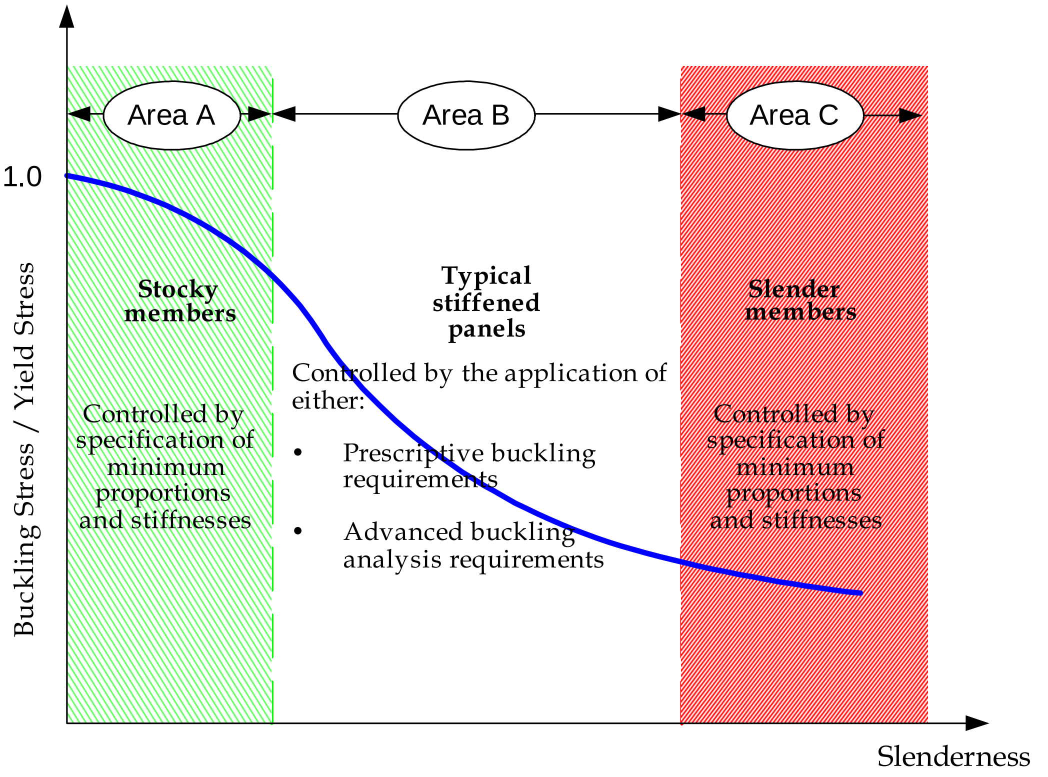
(b) Prescriptive Buckling Requirements (*Section 10/3 of the Rules*). Analytical formulae for assessment of the critical buckling stress for individual structural members in compression (e.g. buckling of plates, pillars, etc.).

(c) Advanced Buckling Analysis (*Section 10/4 of the Rules*). General description of the requirements of the advanced buckling assessment, application and structural modelling principles.

1.1.1.b The buckling and ultimate strength requirements have been tailored to specifically address the probable collapse behaviour of structural components. This is illustrated in *Figure 10.1.a*. This classifies the properties of structural components in order to decide what requirements are most suitable.

1.1.1.c All the buckling and ultimate strength requirements are based on the net thicknesses of structural members.

Figure 10.1.a
Critical buckling stress and slenderness



1.1.2 Stiffness and proportions

1.1.2.a The stiffness and proportions define the minimum values of net thickness or net inertia of structural components. These requirements cover all structural components and give minimum properties for each item. This is particularly applicable to structural components where it is not easy to derive the applied stresses and hence provides a simple minimum requirement based on experience. The minimum requirements control structural components with high slenderness ratios as illustrated by slenderness area "C" in Figure 10.1.a. Typically these components have an elastic buckling capacity less than 50% of the yield stress.

1.1.2.b Some structural components can be classed as stocky and are sufficiently strong or stiff such that it is not necessary to explicitly assess the buckling capability, i.e. they are likely to fail in compression from yield and not buckling. This is illustrated by slenderness area "A" in Figure 10.1.a. An example of this case would be local buckling of flange of local support members, e.g. flanges of longitudinal stiffeners. Typically these components have an elasto-plastic buckling capacity greater than 90% of the yield stress.

1.1.2.c Structural components between the stocky and slender areas, illustrated by slenderness area "B" in Figure 10.1.a, are controlled by the application of prescriptive buckling requirements or the advanced buckling analysis.

1.1.2.d Despite the above classification, all structural components are assessed for buckling using the requirements for areas "A", "B" and "C"

1.1.3 Prescriptive buckling requirements

1.1.3.a The prescriptive buckling requirements are analytical formulae for assessment of the critical buckling stress for individual structural members (e.g. buckling of plates, stiffeners with attached plate, pillars, etc.). The requirements are dependant on the structural members and failure modes being considered.

1.1.3.b In general, the prescriptive buckling capacity formulae for plates and stiffeners defined in *Chapter 6, Section 3 of IACS Common Structural Rules for Bulk Carriers, January 2006* that are relevant to tanker structures are adopted as the main prescriptive buckling requirements in the Rules.

1.1.3.c For the overall buckling of struts, pillars and cross-ties the elastic buckling stress is defined for all relevant failure modes and the critical buckling stress consider Johnson-Ostenfeld correction for elastic buckling stress above 50% of the yield stress.

1.1.4 Advanced buckling analysis

1.1.4.a The advanced buckling analysis method is based on nonlinear analysis techniques which is able to more theoretically predict the complex behaviour of stiffened or un-stiffened panels. Alternative methods equivalent to the standard approach are acceptable provided that the method and results are calibrated to the standard approach, see *Appendix D*.

1.1.4.b General requirements and effects to be considered for the advanced buckling analysis are specified such that any designer can perform an advanced buckling analysis. The application, structural modelling principles and assessment criteria are also specified.

1.1.4.c The Rules allow the use of the ultimate capacity, defined as the maximum load carrying capacity, for certain structural members (e.g. deck, side, bottom) subject to lifetime extreme loading.

1.1.4.d A standard procedure and software that complies with the requirements to the advanced buckling analysis are provided by the Classification Societies. The current version of the standard procedure does not cover the following structural members and hence the Rules apply prescriptive buckling requirements to these members:

(a) Global buckling mode for primary support members

(b) Buckling of web plates in way of openings

(c) Pillars and cross-ties

1.1.5 Symbols and definitions

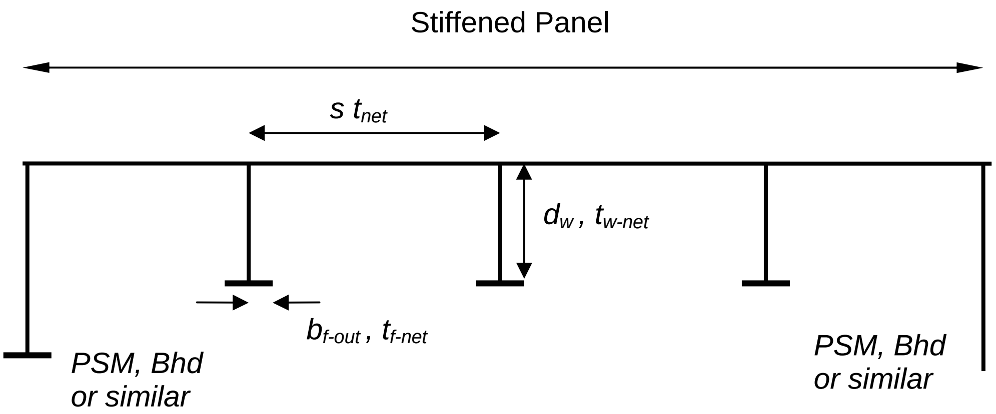
s, s_w, s_{stf}	spacing between stiffeners, in mm
l, l_{stf}	length/span of stiffener, in m
l_a	length of shorter edge of plate panel or as defined in the application, in mm
a	aspect ratio of plate panel
t_{net}	net thickness of plate panel, in mm
t_{w-net}	net thickness of web plate , in mm
t_{f-net}	net thickness of flange/face plate , in mm

d_w	depth of member, in mm
b_f	Breadth of flange/face plate, in mm
b_{f-out}	Breadth of flange outstand, in mm. $b_{f-out} = b_f / 2$ for tee-profiles.
$t_{bkt-net}$	net thickness of bracket , in mm
d_{bkt}	depth of bracket , in mm
l_{bkt}	effective length of edge of bracket, in mm
l_{bdg}	bending span of primary support member, in m
S	distance between primary support members, in m
$A_{w-net50}$	net web area of primary support member, in cm^2
$A_{f-net50}$	net flange area of primary support member, in cm^2
C C_w, C_f	slenderness coefficients for stiffness and proportions
E	modulus of elasticity, 2.06x10^5 N/mm^2
σ_{yd}	specified minimum yield strength of material , in N/mm^2
σ_{xcr} σ_{ycr} τ_{cr}	critical (ultimate) stresses of plate panels, in N/mm^2
C_x C_y C_τ	reduction factors for prescriptive buckling of plate panels
σ_E σ_{ET} σ_{ETF}	lateral/torsional elastic buckling stress for struts, pillars and cross-ties, in N/mm^2
σ_{cr}	critical buckling stress, for struts, pillars and cross-ties, in N/mm^2
A_{net}	net sectional area of a stiffener without attached plating, in cm^2
I_{net}	net moment of inertia of a stiffener including effective width of the attached plating in cm^4
η	utilisation factor
K	Ratio between elastic buckling stress and yield stress, $K = \sigma_E / \sigma_{yd}$

1.1.6 Plate panels and stiffeners

1.1.6.a An overview of the buckling requirements for plate panels and stiffeners is given in *Table 10.1.a.* The numbers in parenthesis refer to the applicable rule sections and sub-sections. The definition of local support member dimensions is illustrated in *Figure 10.1.b.*

Figure 10.1.b
Dimensions of Stiffened Panels and Stiffeners



<table>
<tr><th colspan="5">Table 10.1.a
Requirements for plate panels and stiffeners</th></tr>
<tr><th rowspan="2">Structural members</th><th>Slenderness Area "A"</th><th colspan="2">Slenderness Area "B"</th><th>Slenderness Area "C"</th></tr>
<tr><th>Minimum stiffness and proportions (10/2)</th><th>Prescriptive buckling requirements (10/3)</th><th>Advanced buckling requirements (10/4)</th><th>Minimum stiffness and proportions (10/2)</th></tr>
<tr><td>Plate panels, t_{net}</td><td></td><td>Uni-axial or Shear, critical stress, reduction factor C (3.2.1)</td><td></td><td>Ratio s/t_{net} (2.2.1)</td></tr>
<tr><td rowspan="2">Stiffeners and longitudinals stresses from Longitudinal strength assessment (Section 8/1.4)</td><td rowspan="3">Ratio bf/tf (2.2.1.1)</td><td>Column Buckling Mode (3.3.2., 3.3.4)</td><td></td><td rowspan="3">Ratio dw/tw (2.2.1.1)

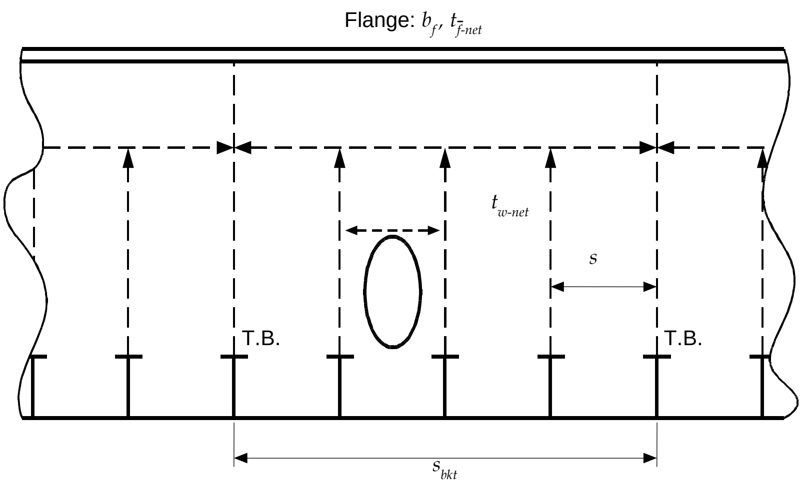
Inertia req I_{net} Column buckling (2.2.2)</td></tr>
<tr><td>Torsional Buckling Mode (3.3.3)</td><td></td></tr>
<tr><td>Stiffened panels Stresses from FEM (Section 9/2)</td><td></td><td>Bi-axial and Shear with Lateral pressure, Buckling or Ultimate capacity (4.1.1)</td></tr>
<tr><td colspan="5"><u>Note</u>
1. Requirement applied to ensure structural member is stocky
2. Numbers in parenthesis are references to the applicable Rule sections
3. Slenderness areas are illustrated in Figure 10.1.a</td></tr>
</table>

1.1.7 Primary support members

1.1.7.a An overview of the buckling requirements for primary support members is given in Table 10.1.b. The definition of primary support member dimensions' is illustrated in Figure 10.1.c.

<table>
<tr><th colspan="5">Table 10.1.b
Requirements for Primary Support Members</th></tr>
<tr><th rowspan="2">Structural members</th><th>Slenderness area "A"</th><th colspan="2">Slenderness area "B"</th><th>Slenderness area "C"</th></tr>
<tr><th>Minimum stiffness and proportions
(10/2)
see Note 1</th><th>Prescriptive buckling requirements
(10/3)</th><th>Advanced buckling requirements
(10/4)</th><th>Minimum stiffness and proportions
(10/2)</th></tr>
<tr><td>Regularly stiffened web panels
see note 4</td><td rowspan="4">Ratio $b_{f\text{-}out}/t_{f\text{-}net}$
(2.3.1)</td><td></td><td>Bi-axial plus shear
Buckling capacity
(4.1.1)</td><td rowspan="4">Ratio
$s_w/t_{w\text{-}net}$
(2.3.1)</td></tr>
<tr><td>Irregularly stiffened or shaped web plate (between stiffeners)</td><td></td><td>Bi-axial plus shear
Buckling capacity
(4.1.1)</td></tr>
<tr><td>Web plate iwo openings</td><td>Uni-axial plus shear
Buckling capacity
(3.4)</td><td></td></tr>
<tr><td>Flange on free edge of PSM</td><td></td><td></td></tr>
<tr><td>Tripping brackets</td><td>Length s_{bkt} between brackets
Torsional buckling
(2.3.3)</td><td></td><td></td><td></td></tr>
<tr><td>Web stiffeners</td><td>Ratio $b_{f\text{-}out}/t_{f\text{-}net}$
(2.2.1)</td><td></td><td></td><td>Ratio $d_w/t_{w\text{-}net}$
(2.2.1)
Inertia req, I_{net}
Column buckling
(2.3.2)</td></tr>
<tr><td>Bending stiffness of PSM</td><td>Inertia req, I_{net50}
Column buckling
(2.3.2)</td><td></td><td></td><td></td></tr>
<tr><td colspan="5"><u>Note:</u>
1. Requirement applied to ensure structural member is stocky
2. Numbers in parenthesis are references to the applicable Rule paragraphs
3. Slenderness areas are illustrated in Figure 10.1.a
4. Applicable for web panels with regularly spaced stiffening, if not then the requirements for Web plate are to be applied</td></tr>
</table>

Figure 10.1.c
Dimensions of primary support members



1.1.8 Other structure

1.1.8.a An overview of the buckling requirements for other structural members such as brackets, edge stiffening, pillars and corrugated bulkheads is given in *Table 10.1.c*.

Table 10.1.c
Requirements for Other Structural members

Structural members	Slenderness area "A"	Slenderness area "B"		Slenderness area "C"
	Minimum stiffness and proportions (*10/2*) *see Note 1*	Prescriptive buckling requirements (*10/3*)	Advanced buckling requirements (*10/4*)	Minimum stiffness and proportions (*10/2*)
Bracket thickness, t_{bkt}	Ratio d_{bkt}/t_{bkt} (*2.2.1*) Maximum edge length without stiffening, l_{bkt} (*2.4.2*)			
Edge reinforcement in way of openings and bracket edges	Min. depth of edge stiffeners., d_w (*2.4.3*)			Slenderness ratios (*2.2.1*)
Struts and Pillars	-	Column buckling and Torsional buckling (*3.5.1*)		Slenderness ratios (*2.2.1*)
Cross Ties	-	Column buckling and Torsional buckling (*3.5.1*)	Bi-axial plus shear Buckling of web plate (*4.1.1*)	-
Corrugated bulkhead plating	-	Uni-axial Buckling of flange (*3.5.2*)	-	Ratio s/t_{net} (*2.2.1*)
Corrugated bulkhead	-	Column buckling (*3.5.2*)	-	-

<u>Note</u>
1. Requirement applied to ensure structural member is stocky
2. Numbers in parenthesis are references to the applicable Rule paragraphs
3. Slenderness areas are illustrated in Figure 10.1.a

2 STIFFNESS AND PROPORTIONS

2.1 Structural Members

2.1.1 General

2.1.1.a The requirements for the minimum proportions of local and primary support members are based on an elastic buckling capacity of plate panels, with an aspect ratio (long edge/short edge) not less than one, given by:

$$\sigma_E = 0.9 C_\sigma E \left(\frac{t_{net}}{1000 l_a} \right)^2 \text{ N/mm}^2$$

$$\tau_E = 0.9 C_\tau E \left(\frac{t_{net}}{1000 l_a} \right)^2 \text{ N/mm}^2$$

2.1.1.b The buckling coefficient is calculated for the critical buckling mode for each structural member and is calibrated at the lower limit of slenderness area "A" and the upper limit of slenderness area "C".

2.1.1.c The Johnson-Ostenfeld correction is used to calculate the critical buckling capacity from the elastic buckling capacity making allowance for the plasticity effects.

2.1.1.d The requirements are based on mild steel with a correction factor for higher material yield strength, an example showing the requirements for the breadth of flange outstands to flange thickness ratio is given below:

$$b_{f-out} / t_{f-net} = 12\sqrt{\frac{235}{\sigma_{yd}}} \quad \Rightarrow \quad t_{f-net} = \frac{b_{f-out}}{12}\sqrt{\frac{\sigma_{yd}}{235}} \quad \text{mm}$$

2.2 Plates and Local Support Members

2.2.1 Proportions of plate panels and local support members

2.2.1.a The requirement for the minimum proportions of plate panels between the stiffeners/longitudinals is a maximum slenderness ratio and is calibrated based on current practice with adjustments for the net thickness approach and using the upper limit of slenderness area "C".

2.2.1.b Similarly, the requirement for the minimum proportions of web plate is a maximum slenderness ratio and is calibrated in the same way.

2.2.1.c The requirement for face plate and flanges is specified such that torsional buckling of the flange is inhibited, noting that torsional buckling of the flange is not covered by other buckling criteria. The requirement is calibrated to give stocky proportions of the face plates, based on existing practice with adjustments for the net thickness approach and using the lower limit of slenderness area "A".

2.2.1.d The proportional requirements are developed based on the assumptions shown in *Table 10. 2.a*.

Table 10.2.a
Proportions for plates and stiffeners – Normal strength steel (σ_{yd} = 235N/mm²)
Comparison based on the assumption of axial compressive stresses

Requirement		F	σ_{EL}	K	λ	σ_{cr}	η	Required slenderness coefficient, C
s/t_{net}	see note 1	4.0	74	0.32	1.78	74	0.32	100
s/t_{net}	see note 2	4.0	47	0.20	2.23	47	0.20	125
$d_w/t_{w\text{-}net}$	L or T	4.0	132	0.56	1.34	130	0.55	75
$d_w/t_{w\text{-}net}$	Bulb	1.0 (1.25)	135 (138)	0.58 (0.59)	1.32 (1.30)	133 (135)	0.57 (0.59)	37 (41)
$d_w/t_{w\text{-}net}$	FB	0.43	165	0.70	1.19	151	0.64	22
$b_{f\text{-}out}/t_{f\text{-}net}$		0.43	554	2.36	0.65	210	0.89	12

Where

F	Buckling edge constraint factor
σ_{EL}	Elastic buckling stress, in N/mm²
K	Ratio between elastic buckling stress and yield stress, $K = \sigma_E/\sigma_{yd}$
λ	Slenderness ratio $\lambda = (\sigma_{yd}/\sigma_E)^{0.5}$
σ_{cr}	Critical buckling stress (Johnson-Ostenfeld correction), in N/mm²
η	Utilisation factor relative to yield, $\eta = \sigma_{cr}/\sigma_{yd}$

Notes
1) Hull envelope and tank boundaries
2) Higher for slenderness for structures such as non-watertight bulkheads, platforms and internal decks in machinery area, accommodations, etc.

(Comment: The values in brackets for bulb are based on proposed change (corrigenda 2), using the torsional buckling strength of 3.3.3 to verify the increased edge constraint factor from 1.0 to 1.25).

The development of the requirement for face plates are shown in detail below:

$$\sigma_E = F\frac{\pi^2 E}{12(1-\nu^2)}\left(\frac{t_{f-net}}{b_{f-out}}\right)^2 \text{ N/mm}^2$$

and

$$\sigma_E = \geq K\sigma_{yd} \text{ N/mm}^2$$

hence

$$\frac{b_{f-out}}{t_{f-net}} \leq \sqrt{\frac{F}{K}\frac{\pi^2}{12(1-\nu^2)}}\cdot\sqrt{\frac{E}{\sigma_{yd}}}$$

$$\leq 0.4\sqrt{\frac{E}{\sigma_{yd}}}$$

hence

$$t_{f-net} = \frac{b_{f-out}}{12}\sqrt{\frac{\sigma_{yd}}{235}} \text{ mm}$$

Where:

F 0.43, F = 0.43 + (s/l)² ≈ 0.43 for simply supported plate

K 2.36

2.2.2 Stiffness of stiffeners

2.2.2.a The purpose of the inertia stiffness requirement for stiffeners is to prevent lateral instability and is based on the Euler buckling formula for a simply supported stiffener. The required inertia stiffness is higher for longitudinals subject to hull girder stresses than for other stiffeners. The criteria will effectively limit the use of flat bars for the deck longitudinals.

$$\sigma_E = \frac{10^{-4}\pi^2 E I_{net}}{l_{stf}^2 \, A_{net}} \quad \text{N/mm}^2$$

and

$$\sigma_E \geq K\,\sigma_{yd}$$

hence

$$\mathrm{I}_{net} = C\,l_{stf}^2\,A_{net}\,\frac{\sigma_{yd}}{235} \qquad \text{cm}^4$$

2.2.2.b The inertia stiffness requirement has been based calibrated based on current practice with adjustments for the net thickness approach to give a slenderness coefficient C of:

1.43. Based on K = 2 for stiffeners subjected to hull girder stresses, which represents a slenderness ratio of λ=0.71

0.72. Based on K = 1 for stiffeners not subject to hull girder stresses, which represents a slenderness ratio of λ=1.0, ie the elastic buckling stress to be equal to the yield stress of the material.

2.2.2.c In deriving the required inertia requirements, an effective breadth of attached plating not exceeding 80% of the total width for cross sectional area and moment of inertia is to be assumed for simplicity. Formulae for calculating the effective plate width are found in several buckling codes. However, adopting such approach was not considered necessary for this simplified approach.

2.2.2.d The reference yield stress is to be taken for the attached plate. The purpose of the stiffener is to stabilize the plate, and the higher yield stress of the plate which allows higher compressive stresses in the panel should result in the higher moment of inertia to keep the panel in shape.

2.3 Primary Support Members (PSM)

2.3.1 Proportions of web plate and flange/face plate

2.3.1.a The requirements for the minimum proportions of PSM are based on the assumptions shown in *Table 10.2.b*. The web spacing to thickness ratio, $s_w/t_{w\text{-}net}$, requirement is the same as for plate panels making up the hull envelope plating or tank boundaries. The breadth of flange outstand to flange thickness ratio, $b_{f\text{-}out}/t_{f\text{-}net}$, for stiffeners is also used for the PSM face flat to ensure that the flange is stocky and hence supports the "free" edge of the PSM.

Table 10.2.b Proportion to Primary Support Members – Mild steel (σ_{yd} = 235N/mm²)							
Requirement	F	σ_{EL}	K	λ	σ_{cr}	η	Required slenderness coefficient, C
web plate $s_w / t_{w\text{-}net}$	4.0	74	0.32	1.78	74	0.32	100
face flat $b_{f\text{-}out} / t_{f\text{-}net}$	0.43	554	2.36	0.65	210	0.89	12

2.3.2 Stiffness requirements

2.3.2.a The purpose of these criteria is to prevent instability of web stiffeners. The criteria are based on Euler buckling equations and consider compressive stresses parallel and normal to the direction of the web stiffening. The criteria are calibrated such that the web stiffeners provide effective support for the web plate and hence the PSM.

2.3.2.b The criterion for web stiffeners parallel to compressive stresses as given in *Section 10, Table 10.2.2 (a)* is identical to local support members, see *2.2.2*.

2.3.2.c The buckling mode for web stiffeners normal to the compressive stress as given in *Section 10, Table 10.2.2 (b)* is more complicated. The criterion is based on *DNV Classification Note 30.1, 1995*. It is assumed that out of plane force in the web plate is related to the thickness of the web. Hence, the web stiffener inertia stiffness requirement to resist the out of plane forces increases proportionally to the web-thickness. This requirement has been calibrated to ensure that the web stiffener elastic buckling capability is higher that the elastic buckling capability of the web plate. In case of slender web plates, this criterion will also provide a higher elastic buckling capability in the web stiffener than the ultimate capacity (advanced buckling method) of the web plate.

2.3.2.d The purpose of the overall stiffness criterion (*2.3.2.3*) is to ensure that the transverse primary support members have sufficient stiffness to ensure that axially compressed longitudinals are effectively supported. The criteria controls global lateral instability of the PSM and is based on *S. P. Timonshenko and J. M. Gere "Theory of Elastic Stability"* and calibrated with current practice.

2.3.3 Spacing between flange supports or tripping brackets

2.3.3.a The purpose of this requirement is to prevent torsional buckling of primary support members. This is the only requirement covering the torsional buckling mode of PSM and hence the requirement is calibrated to ensure that the flanges are stocky. The requirement is given below.

$$s_{bkt} \le b_f C \sqrt{\frac{A_{f-net50}}{\left(A_{f-net50} + \frac{A_{w-net50}}{3}\right)} \frac{235}{\sigma_{yd}}}$$ m, but need not be less than $s_{bkt\text{-}min}$

Where:

b_f breadth of flange, in mm

C	slenderness coefficient: 0.022 symmetrical flanges 0.033 for one-sided flanges
$A_{f\text{-}net50}$	cross sectional area of flange/face plate, in cm²
$A_{w\text{-}net50}$	cross sectional area of the web plate, in cm²
σ_{yd}	specified minimum yield stress of the material, in N/mm²
$s_{bkt\text{-}min}$	= 3.0 m for primary support members in the cargo tank region, on tank boundaries or on hull envelope including external decks = 4.0 m for primary support members in other areas

2.3.3.b The correction for web-area ($A_f/(A_f+0.33A_w)$) will require a smaller distance between tripping brackets for primary support members with a large web depth to flange area ratio. The requirement is based on the torsional buckling strength criteria in *Chapter 8.5 of DNV Offshore Standard, DNV-RP-C201, October 2002 "Buckling Strength of Plated Structures*". The slenderness coefficients provide a torsional buckling capacity σ_T as follows:

For symmetrical flanges, C=0.022 => σ_T = 0,85 σ_{yd}

For one sided flanges, C=0.033 => σ_T = 0.96 σ_{yd}

It has been the intention to keep one sided flanges more stocky than the symmetrical ones, due to the un-symmetric bending behaviour of the members with one sided flanges

2.3.3.c Tripping brackets need not be spaced closer than 3m on PSM supporting tank or external boundaries, i.e. subjected to tank pressures or sea pressures, based on *ABS Rules 2002* and current practice.

2.3.3.d Based on current experience, tripping brackets in other areas need not be spaced closer than 4 m, e.g. in way of engine room and superstructure (excl. in way of tank boundaries and external boundaries). For such PSM, which usually have small flange area and moderate stress level, $s_{bkt\text{-}min}$= 4.0 has been considered to be sufficient. In addition the stiffening between tripping brackets will usually contribute with some tripping resistance for these primary support members.

2.4 Other Structure

2.4.1 Proportions of pillars

2.4.1.a Local buckling of the web and flanges of a pillar cross section are controlled by limiting the slenderness proportional requirements.

2.4.2 Proportions of brackets

2.4.2.a The criteria are based on *DNV Rules 2002, Pt.3 Ch1. Sec. 3 C202*. These Rules specify that the length of the free edge of brackets is not to exceed 50 times the thickness, calculated as gross thickness minus the corrosion addition specified in the DNV Rules. In way of a ballast tanks this corrosion addition is typically 1.5 mm which is 50% of t_{corr} as defined in *Section 6/3*. Assuming 20% wastage allowance applied in the current Rules, this ratio to be 55 (10% increase from 50) for net scantlings as defined in CSR Rules. Based on a standard end bracket having a base angle, α, of

90 degrees (see *Figure 10.2.a*) and with edge length/net thickness ratio of 55, a number of other bracket geometries with same buckling strength have been found, see *2.4.2b*.

2.4.2.b The minimum thickness of end brackets satisfying the given buckling requirements is given. The buckling formulation is based on the following assumptions, see also *Table 10.2.c* and *Figure 10.2.a*:

- limiting the depth to thickness ratio of end brackets without edge stiffening to ensure brackets are stocky i.e. in slenderness area "A"
- the triangular end bracket is idealised as a rectangular plate with short and long edge taken as 2/3 of the length of free edge and 2/3 of the depth of the bracket, respectively
- the free edge of brackets without edge stiffening is allowed to move out of plane
- the loading pattern is triangular
- the thickness slenderness coefficient is applicable for a range off base angles (50º < α < 150º). Studies have confirmed this to be adequate
- End brackets with edge reinforcements are based on the same assumptions as for end brackets without edge reinforcement. The buckling coefficient for brackets with edge reinforcement assumes that the edge reinforcement is sufficiently stiff to prevent buckling of the bracket edge (simply supported edge).

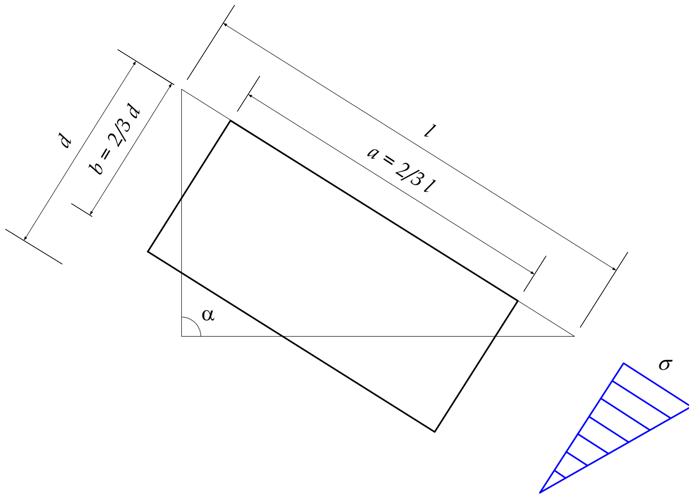
2.4.2.c Since this is a buckling requirement for brackets subjected to compression at edge, it is not relevant for brackets only subjected to tensile stresses, e.g. internal brackets in a tank surrounded by void spaces, and hence this buckling requirement is not applicable for these brackets. However other requirements to bracketed connections will apply to such brackets, see *Section 4 of the Rules*.

Table 10.2.c
Proportions of brackets – Normal strength steel (σ_{yd} = 235N/mm²)

Requirement	F	σ_{EL}	K	λ	σ_{cr}	η	Required slenderness coefficient, C
Without edge reinforcement	0.90	498	2.12	0.69	207	0.88	$20\left(\frac{d}{l}\right)+16$ where $0.25 \le \left(\frac{d}{l}\right) \le 1/0$
With edge reinforcement	7.64	650	2.77	0.60	214	0.91	70

Where
F buckling edge constraint factor based on the assumptions in 2.4.2.b

Figure 10.2.a
Definition of brackets



2.4.2.d Where the length of the edge of tripping brackets exceeds 75 times the net thickness, then the free edge is to be stiffened by a flange or edge stiffener. For a connection bracket this ratio is 50-55, however the stress level in the middle of a tripping brackets is lower than for end brackets, as end brackets will have maximum compression near the midpoint of the edge whereas tripping brackets act as a cantilever and hence the maximum compression is close to the support. DNV Rules (2002) have a value of 60, based on gross scantlings. For a typical tripping bracket in a cargo tank, the gross thickness might be 12.5mm which corresponds to a net thickness of 12.5-2.5=10mm (20-25% corrosion allowance) and hence the coefficient for net scantlings becomes: 60x12.5/10 = 75.

2.4.3 Requirements to edge reinforcements in way of openings and bracket edges

2.4.3.a The requirement for the moment of inertia of edge reinforcement stiffeners in way of openings or cut outs is based on the Euler buckling formula for a simply supported stiffener. It is assumed that only sectional properties of the edge reinforcement itself are used and that the edge reinforcement is a flat bar stiffener, excluding the effective web plate flange which is a conservative assumption.

$$\sigma_E = \frac{10^{-4}\pi^2 E}{l_{stf}^{\ 2}} k_{net}^{\ 2} \ \text{N/mm}^2$$

$$\sigma_E \geq K\sigma_{yd}$$

hence

$$k_{net} \geq 100\frac{\sqrt{K}}{\pi} l_{stf}\sqrt{\frac{\sigma_{yd}}{E}} \quad \text{cm}$$

the radius of gyration, k_{net} for an flat bar stiffener is

$$k_{net} = \sqrt{\frac{I_{net}}{A}} = \frac{d_w}{20\sqrt{3}} \ \text{cm}$$

rearranging the above gives

$$d_w \geq \frac{2000\sqrt{3K}}{\pi} l_{stf}\sqrt{\frac{\sigma_{yd}}{E}} \quad \text{mm}$$

2.4.3.b The requirement for the minimum depth of end bracket edge stiffening is calibrated to give a stocky slenderness ratio, noting that the highest compressive stress occurring at the midspan, and hence K is taken as 4, where K is the elastic buckling strength to yield strength ratio. This gives:

$$d_w \geq 2200\, l_{stf} \sqrt{\frac{\sigma_{yd}}{E}} = 75\, l_{stf} \sqrt{\frac{\sigma_{yd}}{235}} \quad \text{mm}$$

2.4.3.c The requirement for the minimum depth of edge stiffening of tripping brackets or openings takes into account the lower stress level at the midspan of the edge stiffener and hence K is taken as 2, which gives:

$$d_w \geq 1560\, l_{stf} \sqrt{\frac{\sigma_{yd}}{E}} = 50\, l_{stf} \sqrt{\frac{\sigma_{yd}}{235}} \quad \text{mm}$$

2.4.3.d A minimum depth of 50mm for the edge stiffener is found reasonable and comparable with existing practice.

3 PRESCRIPTIVE BUCKLING REQUIREMENTS

3.1 General

3.1.1 Scope

3.1.1.a This section summarises the background for determination of the critical buckling stress, definitions of buckling utilisation factors and other measures necessary to control buckling of local support members and primary support members.

3.1.1.b Chapters *3.2* and *3.3* are based on the uni-axial compressive strength of plane plates and stiffeners in *Chapter 6, Section 3 of IACS Common Structural Rules for Bulk Carriers, January 2006*,. Only those relevant to tanker structures are adopted into CSR/tanker rules.

3.2 Buckling of Plates

3.2.1 Uni-axial buckling of plates

3.2.1.a The critical buckling stress for plate panels are taken as the ultimate compressive stress for rectangular plates subjected to uni-axial membrane compressive stresses or shear stress, shown in column "reduction factor C" of Table 10.3.1.

3.2.1.b For transverse compression the correction factor, F1 in Table 1 of *Chapter 6, Section 3 of IACS Common Structural Rules for Bulk Carriers, January 2006*, is set to 1.0. This is based on comparison studies with advanced buckling method as well as non-linear FE calculations.

3.3 Buckling of stiffeners

3.3.1 Critical buckling stress

3.3.1.a The buckling of stiffeners and longitudinals is to be checked for the ultimate compressive strength in column and torsional buckling modes.

3.3.2 Column buckling mode

3.3.2.a It is considered that for this topic, no information in addition to that shown in the Rules, is necessary to explain the background.

3.3.3 Torsional buckling mode

3.3.3.a It is considered that for this topic, no information in addition to that shown in the Rules, is necessary to explain the background.

3.4 Primary Support Members

3.4.1 Buckling of web plate of primary support members in way of opening

3.4.1.a The formulae for plate buckling are developed for regular geometry and idealized stress patterns, whereas the geometry and stress gradients around openings and cut-outs are rather complex. This complexity is taken into account by adoption of conservative assumptions for the formulae for plate buckling.

3.4.1.b The important stress components for buckling control of the web plate in way of openings are the axial or tangential stress flow passing the hole and the shear stress.

These stress components used in the buckling criterion should account for the stress increase due to the presence of the opening, See *10/Figure 10.3.3*. The normal stress component acting perpendicular to the opening is not considered critical for buckling and may be neglected in the buckling assessment.

3.4.1.c For openings without edge reinforcements, the calculation of the separate critical buckling compression stress, $C\sigma_{yd}$, assumes a plate buckling model of the web plate area in way of the opening with three edges simply supported and one edge free (towards the opening), see *Figure 10.3.a*

3.4.1.d For opening with edge reinforcements, the calculation of the critical buckling stress, $C\sigma_{yd}$,, assumes a plate buckling model with all four edges simply supported.

3.4.1.e The calculation of the separate critical shear buckling stresses, $C_\tau\sigma_{yd}/\sqrt{3}$, assumes a plate buckling model comprising the web panel including the opening with all four edges simply supported if the opening is not fitted with edge stiffeners, then Case 6 is assumed. For the situation where the openings are fitted with edge stiffeners, then a plate buckling model comprising the web plate area in way of the opening and Case 5 is assumed. Cases 5 and 6 are defined in *Section 10/Table 10.3.1*.

3.4.1.f The basis of the buckling interaction formula is given in *Chapter 6, Section 3 of IACS Common Structural Rules for Bulk Carriers, January 2006*, the following interaction formula is applicable for the stress combination of uni-axial compression and shear:

$$\eta = \left(\frac{|\sigma_{av}|}{C\sigma_{yd}} \right)^{e} + \left(\frac{|\tau_{av}|\sqrt{3}}{C_\tau \sigma_{yd}} \right)^{e_\tau}$$

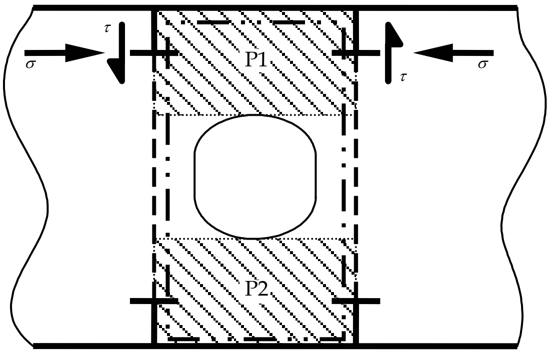
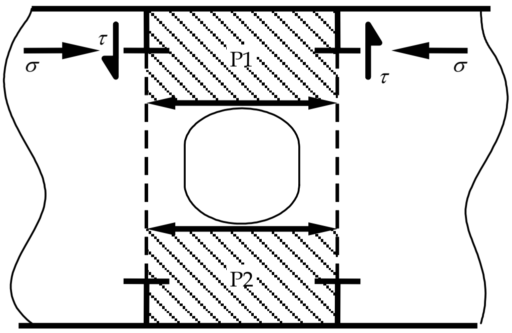
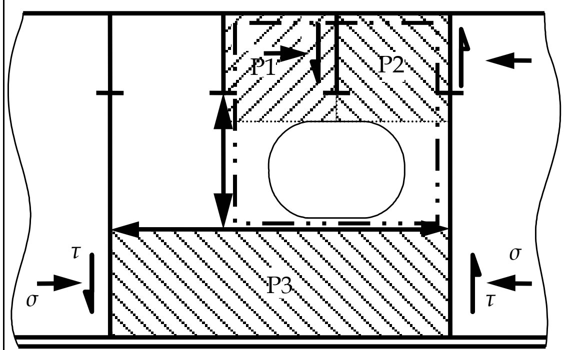
Where:

σ_{av}	the average compressive stress in the area of web plate being considered according to Case 1, 2 or 3 as appropriate in *10/Table 10.3.1*, in N/mm²
τ_{av}	the average shear stress in the area of web plate being considered according to Case 5 or 6 in *10/Table 10.3.1*, in N/mm²
$e = 1 + C^4$	exponent for compressive stress
$e_\tau = 1 + C \cdot C_\tau^2$	exponent for shear stress
$C = C_x$	reduction factor according to Case 1 or 3, *10/Table 10.3.1*
$C = C_y$	reduction factor according to Case 2, *10/Table 10.3.1*
C_τ	reduction factor according to Case 5 or 6, *10/Table 10.3.1*

3.4.1.g Further details about the cases to be used in way of PSM openings are shown in *Table 10.3.a.*

3.4.1.h The buckling control is given by criteria:

$\eta \le \eta_{allow}$

<table>
<tr><th colspan="3">Table 10.3.a
Buckling of web plate of primary support members in way of opening</th></tr>
<tr><td rowspan="2">Mode</td><td colspan="2">Reduction Factors</td></tr>
<tr><td>C_x, C_y</td><td>C_τ</td></tr>
<tr><td>(a) without edge reinforcements
</td><td>Separate reduction factors are to be applied to areas P1 and P2 using Case 3, 10/Table 10.3.1, with edge stress ratio:
$\psi = 1.0$</td><td>A common reduction factor is to be applied to areas P1 and P2 using Case 6, 10/Table 10.3.1 for area marked:
</td></tr>
<tr><td>(b) with edge reinforcements
</td><td>Separate reduction factors are to be applied for areas P1 and P2 using:
C_x for Case 1 or C_y, for Case 2,
see 10/Table10.3.1
with stress ratio
$\psi = 1.0$</td><td>Separate reduction factors are to be applied for areas P1 and P2 using Case 5, 10/Table 10.3.1</td></tr>
<tr><td>(c) example of hole in web
</td><td colspan="2">Panels P1 and P2 are to be evaluated in accordance with (a). Panel P3 is to be evaluated in accordance with (b)</td></tr>
<tr><td colspan="3"><u>Note</u>
1. Web panels to be considered for buckling in way of openings are shown shaded and numbered P1, P2, etc.</td></tr>
</table>

3.5 Other Structures

3.5.1 Struts, pillars and cross-ties

3.5.1.a Buckling requirements are given for pillar type structures subject to axial loading only and these address the following global buckling modes:

(a) Column buckling (flexural buckling). Bending about the axis of least resistance of the cross section. This may be the critical buckling mode of slender pillars with double symmetrical cross sections or pillars not susceptible to twisting.

(b) Torsional buckling. Twisting of cross section without bending. This buckling mode may be critical for some open, thin walled cross sections in which the shear centre and the centroid coincide.

(c) Column (flexural)-torsional buckling. Simultaneous twisting and bending of cross section. This buckling mode is only relevant for cross section whose shear centre and the centroid do not coincide and which are torsionally weak.

3.5.1.b The formulae for the elastic buckling is based on *"Buckling of Bars, Plates and Pillars", Brush and Almroth, McGraw-Hill 1975*. End constraint factors for calculation of effective span of the pillars are also considered.

3.5.1.c For elastic buckling stresses exceeding 50% of stresses exceeding specified minimum yield stress of the material, the critical compressive stress for global buckling is derived using the Johnson-Ostenfeld plasticity correction factor.

3.5.1.d Local buckling of the thin-walled part of the cross section is covered by slenderness requirements shown in *Section 10/2*. In the formulae for the global buckling modes it is assumed that the cross sections are 100% effective.

3.5.1.e Local plate buckling of cross-ties is also checked using the advanced buckling method as part of the cargo tank FEM strength verification procedure.

3.5.1.f Global buckling due to bending moments from lateral pressure are not considered

3.5.2 Corrugated bulkheads

3.5.2.a Local buckling of the corrugation flange is to be checked for uni-axial plate buckling according to *3.2*.

3.5.2.b Global buckling of corrugated bulkheads is to be checked as a pillar/strut according to *3.5.1*.

3.5.2.c The column buckling mode is applicable for the global buckling of corrugations subject to axial compression (e.g. longitudinal bulkheads with horizontal corrugations). This failure mode is normally not critical for vertically corrugated tank bulkheads.

4 ADVANCED BUCKLING ANALYSES

4.1 General

4.1.1 Assessment

4.1.1.a Details of the advanced buckling analysis are given in *Appendix D of the Rules.*