Buckling Assessment of Plated Structures

January 2020

Rule Note
NI 615 DT R03 E

Marine & Offshore
92937 Paris la Défense Cedex – France
Tel: + 33 (0)1 55 24 70 00
Website: http://www.veristar.com
Email: veristarinfo@bureauveritas.com

BUREAU VERITAS MARINE & OFFSHORE

GENERAL CONDITIONS

1. INDEPENDENCE OF THE SOCIETY AND APPLICABLE TERMS

1.1 The Society shall remain at all times an independent contractor and neither the Society nor any of its officers, employees, servants, agents or subcontractors shall be or act as an employee, servant or agent of any other party hereto in the performance of the Services.

1.2 The operations of the Society in providing its Services are exclusively conducted by way of random inspections and do not, in any circumstances, involve monitoring or exhaustive verification.

1.3 The Society acts as a services provider. This cannot be construed as an obligation bearing on the Society to obtain a result or as a warranty. The Society is not and may not be considered as an underwriter, broker in Unit's sale or chartering, expert in Unit's valuation, consulting engineer, controller, naval architect, designer, manufacturer, shipbuilder, repair or conversion yard, charterer or shipowner; none of them above listed being relieved of any of their expressed or implied obligations as a result of the interventions of the Society.

1.4 The Society only is qualified to apply and interpret its Rules.

1.5 The Client acknowledges the latest versions of the Conditions and of the applicable Rules applying to the Services' performance.

1.6 Unless an express written agreement is made between the Parties on the applicable Rules, the applicable Rules shall be the Rules applicable at the time of entering into the relevant contract for the performance of the Services.

1.7 The Services' performance is solely based on the Conditions. No other terms shall apply whether express or implied.

2. DEFINITIONS

2.1 **"Certificate(s)"** means classification or statutory certificates, attestations and reports following the Society's intervention.

2.2 **"Certification"** means the activity of certification in application of national and international regulations or standards, in particular by delegation from different governments that can result in the issuance of a Certificate.

2.3 **"Classification"** means the classification of a Unit that can result or not in the issuance of a classification Certificate with reference to the Rules. Classification is an appraisement given by the Society to the Client, at a certain date, following surveys by its surveyors on the level of compliance of the Unit to the Society's Rules or to the documents of reference for the Services provided. They cannot be construed as an implied or express warranty of safety, fitness for the purpose, seaworthiness of the Unit or of its value for sale, insurance or chartering.

2.4 **"Client"** means the Party and/or its representative requesting the Services.

2.5 **"Conditions"** means the terms and conditions set out in the present document.

2.6 **"Industry Practice"** means international maritime and/or offshore industry practices.

2.7 **"Intellectual Property"** means all patents, rights to inventions, utility models, copyright and related rights, trade marks, logos, service marks, trade dress, business and domain names, rights in trade dress or get-up, rights in goodwill or to sue for passing off, unfair competition rights, rights in designs, rights in computer software, database rights, topography rights, moral rights, rights in confidential information (including know-how and trade secrets), methods and protocols for Services, and any other intellectual property rights, in each case whether capable of registration, registered or unregistered and including all applications for and renewals, reversions or extensions of such rights, and all similar or equivalent rights or forms of protection in any part of the world.

2.8 **"Parties"** means the Society and Client together.

2.9 **"Party"** means the Society or the Client.

2.10 **"Register"** means the public electronic register of ships updated regularly by the Society.

2.11 **"Rules"** means the Society's classification rules and other documents. The Society's Rules take into account at the date of their preparation the state of currently available and proven technical minimum requirements but are not a standard or a code of construction neither a guide for maintenance, a safety handbook or a guide of professional practices, all of which are assumed to be known in detail and carefully followed at all times by the Client.

2.12 **"Services"** means the services set out in clauses 2.2 and 2.3 but also other services related to Classification and Certification such as, but not limited to: ship and company safety management certification, ship and port security certification, maritime labour certification, training activities, all activities and duties incidental thereto such as documentation on any supporting means, software, instrumentation, measurements, tests and trials on board. The Services are carried out by the Society according to the applicable referential and to the Bureau Veritas' Code of Ethics. The Society shall perform the Services according to the applicable national and international standards and Industry Practice and always on the assumption that the Client is aware of such standards and Industry Practice.

2.13 **"Society"** means the classification society **'Bureau Veritas Marine & Offshore SAS'**, a company organized and existing under the laws of France, registered in Nanterre under number 821 131 844, or any other legal entity of Bureau Veritas Group as may be specified in the relevant contract, and whose main activities are Classification and Certification of ships or offshore units.

2.14 **"Unit"** means any ship or vessel or offshore unit or structure of any type or part of it or system whether linked to shore, river bed or sea bed or not, whether operated or located at sea or in inland waters or partly on land, including submarines, hovercrafts, drilling rigs, offshore installations of any type and of any purpose, their related and ancillary equipment, subsea or not, such as well head and pipelines, mooring legs and mooring points or otherwise as decided by the Society.

3. SCOPE AND PERFORMANCE

3.1 Subject to the Services requested and always by reference to the Rules, the Society shall:

- review the construction arrangements of the Unit as shown on the documents provided by the Client;
- conduct the Unit surveys at the place of the Unit construction;
- class the Unit and enter the Unit's class in the Society's Register;
- survey the Unit periodically in service to note whether the requirements for the maintenance of class are met.

The Client shall inform the Society without delay of any circumstances which may cause any changes on the conducted surveys or Services.

3.2 The Society will not:

- declare the acceptance or commissioning of a Unit, nor its construction in conformity with its design, such activities remaining under the exclusive responsibility of the Unit's owner or builder;
- engage in any work relating to the design, construction, production or repair checks, neither in the operation of the Unit or the Unit's trade, neither in any advisory services, and cannot be held liable on those accounts.

4. RESERVATION CLAUSE

4.1 The Client shall always: (i) maintain the Unit in good condition after surveys; (ii) present the Unit for surveys; and (iii) inform the Society in due time of any circumstances that may affect the given appraisement of the Unit or cause to modify the scope of the Services.

4.2 Certificates are only valid if issued by the Society.

4.3 The Society has entire control over the Certificates issued and may at any time withdraw a Certificate at its entire discretion including, but not limited to, in the following situations: where the Client fails to comply in due time with instructions of the Society or where the Client fails to pay in accordance with clause 6.2 hereunder.

4.4 The Society may at times and at its sole discretion give an opinion on a design or any technical element that would 'in principle' be acceptable to the Society. This opinion shall not presume on the final issuance of any Certificate or on its content in the event of the actual issuance of a Certificate. This opinion shall only be an appraisal made by the Society which shall not be held liable for it.

5. ACCESS AND SAFETY

5.1 The Client shall give to the Society all access and information necessary for the efficient performance of the requested Services. The Client shall be the sole responsible for the conditions of presentation of the Unit for tests, trials and surveys and the conditions under which tests and trials are carried out. Any information, drawing, etc. required for the performance of the Services must be made available in due time.

5.2 The Client shall notify the Society of any relevant safety issue and shall take all necessary safety-related measures to ensure a safe work environment for the Society or any of its officers, employees, servants, agents or subcontractors and shall comply with all applicable safety regulations.

6. PAYMENT OF INVOICES

6.1 The provision of the Services by the Society, whether complete or not, involve, for the part carried out, the payment of fees thirty (30) days upon issuance of the invoice.

6.2 Without prejudice to any other rights hereunder, in case of Client's payment default, the Society shall be entitled to charge, in addition to the amount not properly paid, interests equal to twelve (12) months LIBOR plus two (2) per cent as of due date calculated on the number of days such payment is delinquent. The Society shall also have the right to withhold Certificates and other documents and/or to suspend or revoke the validity of Certificates.

6.3 In case of dispute on the invoice amount, the undisputed portion of the invoice shall be paid and an explanation on the dispute shall accompany payment so that action can be taken to solve the dispute.

7. LIABILITY

7.1 The Society bears no liability for consequential loss. For the purpose of this clause consequential loss shall include, without limitation:

- Indirect or consequential loss;
- Any loss and/or deferral of production, loss of product, loss of use, loss of bargain, loss of revenue, loss of profit or anticipated profit, loss of business and business interruption, in each case whether direct or indirect.

The Client shall defend, release, save, indemnify, defend and hold harmless the Society from the Client's own consequential loss regardless of cause.

7.2 Except in case of wilful misconduct of the Society, death or bodily injury caused by the Society's negligence and any other liability that could not be, by law, limited, the Society's maximum liability towards the Client is limited to one hundred and fifty per-cents (150%) of the price paid by the Client to the Society for the Services having caused the damage. This limit applies to any liability of whatsoever nature and howsoever arising, including fault by the Society, breach of contract, breach of warranty, tort, strict liability, breach of statute.

7.3 All claims shall be presented to the Society in writing within three (3) months of the completion of Services' performance or (if later) the date when the events which are relied on were first discovered by the Client. Any claim not so presented as defined above shall be deemed waived and absolutely time barred.

8. INDEMNITY CLAUSE

8.1 The Client shall defend, release, save, indemnify and hold harmless the Society from and against any and all claims, demands, lawsuits or actions for damages, including legal fees, for harm or loss to persons and/or property tangible, intangible or otherwise which may be brought against the Society, incidental to, arising out of or in connection with the performance of the Services (including for damages arising out of or in connection with opinions delivered according to clause 4.4 above) except for those claims caused solely and completely by the gross negligence of the Society, its officers, employees, servants, agents or subcontractors.

9. TERMINATION

9.1 The Parties shall have the right to terminate the Services (and the relevant contract) for convenience after giving the other Party thirty (30) days' written notice, and without prejudice to clause 6 above.

9.2 In such a case, the Classification granted to the concerned Unit and the previously issued Certificates shall remain valid until the date of effect of the termination notice issued, subject to compliance with clause 4.1 and 6 above.

9.3 In the event where, in the reasonable opinion of the Society, the Client is in breach, or is suspected to be in breach of clause 16 of the Conditions, the Society shall have the right to terminate the Services (and the relevant contracts associated) with immediate effect.

10. FORCE MAJEURE

10.1 Neither Party shall be responsible or liable for any failure to fulfil any term or provision of the Conditions if and to the extent that fulfilment has been delayed or temporarily prevented by a force majeure occurrence without the fault or negligence of the Party affected and which, by the exercise of reasonable diligence, the said Party is unable to provide against.

10.2 For the purpose of this clause, force majeure shall mean any circumstance not being within a Party's reasonable control including, but not limited to: acts of God, natural disasters, epidemics or pandemics, wars, terrorist attacks, riots, sabotages, impositions of sanctions, embargoes, nuclear, chemical or biological contaminations, laws or action taken by a government or public authority, quotas or prohibition, expropriations, destructions of the worksite, explosions, fires, accidents, any labour or trade disputes, strikes or lockouts.

11. CONFIDENTIALITY

11.1 The documents and data provided to or prepared by the Society in performing the Services, and the information made available to the Society, are treated as confidential except where the information:

- is properly and lawfully in the possession of the Society;
- is already in possession of the public or has entered the public domain, otherwise than through a breach of this obligation;
- is acquired or received independently from a third party that has the right to disseminate such information;
- is required to be disclosed under applicable law or by a governmental order, decree, regulation or rule or by a stock exchange authority (provided that the receiving Party shall make all reasonable efforts to give prompt written notice to the disclosing Party prior to such disclosure.

11.2 The Parties shall use the confidential information exclusively within the framework of their activity underlying these Conditions.

11.3 Confidential information shall only be provided to third parties with the prior written consent of the other Party. However, such prior consent shall not be required when the Society provides the confidential information to a subsidiary.

11.4 Without prejudice to sub-clause 11.1, the Society shall have the right to disclose the confidential information if required to do so under regulations of the International Association of Classifications Societies (IACS) or any statutory obligations.

12. INTELLECTUAL PROPERTY

12.1 Each Party exclusively owns all rights to its Intellectual Property created before or after the commencement date of the Conditions and whether or not associated with any contract between the Parties.

12.2 The Intellectual Property developed by the Society for the performance of the Services including, but not limited to drawings, calculations, and reports shall remain the exclusive property of the Society.

13. ASSIGNMENT

13.1 The contract resulting from to these Conditions cannot be assigned or transferred by any means by a Party to any third party without the prior written consent of the other Party.

13.2 The Society shall however have the right to assign or transfer by any means the said contract to a subsidiary of the Bureau Veritas Group.

14. SEVERABILITY

14.1 Invalidity of one or more provisions does not affect the remaining provisions.

14.2 Definitions herein take precedence over other definitions which may appear in other documents issued by the Society.

14.3 In case of doubt as to the interpretation of the Conditions, the English text shall prevail.

15. GOVERNING LAW AND DISPUTE RESOLUTION

15.1 These Conditions shall be construed and governed by the laws of England and Wales.

15.2 The Parties shall make every effort to settle any dispute amicably and in good faith by way of negotiation within thirty (30) days from the date of receipt by either one of the Parties of a written notice of such a dispute.

15.3 Failing that, the dispute shall finally be settled under the Rules of Arbitration of the Maritime Arbitration Chamber of Paris ("CAMP"), which rules are deemed to be incorporated by reference into this clause. The number of arbitrators shall be three (3). The place of arbitration shall be Paris (France). The Parties agree to keep the arbitration proceedings confidential.

16. PROFESSIONAL ETHICS

16.1 Each Party shall conduct all activities in compliance with all laws, statutes, rules, economic and trade sanctions (including but not limited to US sanctions and EU sanctions) and regulations applicable to such Party including but not limited to: child labour, forced labour, collective bargaining, discrimination, abuse, working hours and minimum wages, anti-bribery, anti-corruption, copyright and trademark protection, personal data protection (https://personaldataprotection.bureauveritas.com/privacypolicy).

Each of the Parties warrants that neither it, nor its affiliates, has made or will make, with respect to the matters provided for hereunder, any offer, payment, gift or authorization of the payment of any money directly or indirectly, to or for the use or benefit of any official or employee of the government, political party, official, or candidate.

16.2 In addition, the Client shall act consistently with the Bureau Veritas' Code of Ethics.
https://group.bureauveritas.com/group/corporate-social-responsibility

GUIDANCE NOTE NI 615

Buckling Assessment of Plated Structures

January 2020

Appendix 2 Method Selection for Direct Strength Analysis of Panels

SECTION 1 GENERAL

1 General

1.1 Application

1.1.1 This Guidance Note contains the strength criteria for buckling and ultimate strength of local supporting members, primary supporting members and other structures such as pillars, corrugated bulkheads and brackets.

1.1.2 This Guidance Note is to be applied for buckling of plated structures when it is referred to in the applicable Rules.

1.2 Assumption

1.2.1 For each structural member, the characteristic buckling strength is to be taken as the most unfavourable/critical buckling failure mode.

1.2.2 Unless otherwise specified, the scantling requirements of structural members in this Guidance Note are based on net scantling.

1.2.3 In this Guidance Note, compressive and shear stresses are to be taken as positive, tension stresses are to be taken as negative.

1.3 Scope

1.3.1 The buckling checks are to be performed according to:

- Sec 2 for the slenderness requirements of plates, longitudinal and transverse stiffeners, primary supporting members and brackets
- Sec 3 for the prescriptive buckling requirements of plates, longitudinal and transverse stiffeners, primary supporting members and other structures
- Sec 4 for the buckling requirements of the FE analysis for the plates, stiffened panels and other structures
- Sec 5 for the buckling capacity of prescriptive and FE buckling requirements.

1.3.2 Stiffeners

The buckling check of the stiffeners referred to in this Guidance Note is applicable to the stiffeners fitted along the long edge of the buckling panel.

1.3.3 Enlarged stiffeners

Enlarged stiffeners, with or without web stiffening, used for Permanent Means of Access (PMA) are to comply with the following requirements:

a) slenderness requirements for primary supporting members:
   - for enlarged stiffener web, see Sec 2, [4.1.1], item a)
   - for enlarged stiffener flange, see Sec 2, [4.1.1], item b) and Sec 2, [5.1]
   - for stiffeners fitted on enlarged stiffener web, see Sec 2, [3.1.1].

b) buckling strength of prescriptive requirements:
   - for enlarged stiffener web, see Sec 3, [3.2]
   - for stiffeners fitted on enlarged stiffener web, see Sec 3, [3.1] and Sec 3, [3.3].

c) all structural elements used for PMA are to be complied with for the buckling requirements of the FE analysis in Sec 4 when applicable

d) buckling strength of longitudinal PMA platforms without stiffeners fitted on enlarged stiffener web is to be checked using the criteria for local supporting members in Sec 3, [3.1] and Sec 3, [3.3].

2 Definitions

2.1 General

2.1.1 Buckling definition

'Buckling' is used as a generic term to describe the strength of structures, generally under in-plane compressions and/or shear and lateral loads. The buckling strength or capacity can take into account the internal redistribution of loads depending on the load situation, slenderness and type of structure.

2.1.2 Buckling capacity

Buckling capacity based on this principle gives a lower bound estimate of ultimate capacity, or the maximum load the panel can carry without suffering major permanent set.

Buckling capacity assessment utilises the positive elastic post-buckling effect for plates and accounts for load redistribution between the structural components, such as between plating and stiffeners. For slender structures, the capacity calculated using this method is typically higher than the ideal elastic buckling stress (minimum Eigen value). Accepting elastic buckling of structural components in slender stiffened panels implies that large elastic deflections and reduced in-plane stiffness will occur at higher buckling utilisation levels.

2.1.3 Assessment methods

The buckling assessment is carried out according to one of the two following methods, taking into account different boundary condition types:

- Method A:

  All the edges of the elementary plate panel are forced to remain straight (but free to move in the in-plane directions) due to the surrounding structure/neighbouring plates. The elementary plate is integrated in the structure, which means that it is surrounded by plates that give a strong in plane support. A typical example is a double bottom girder supporting a longitudinal bulkhead.

- Method B:

  The edges of the elementary plate panel are not forced to remain straight due to low in-plane stiffness at the edges and/or no surrounding structure/neighbouring plates. The elementary plate is not surrounded by plates which means that the in-plane support is weak. A typical example is a double bottom girder not supporting a longitudinal bulkhead.

2.2 Buckling utilisation factor

2.2.1 The utilisation factor η is defined as the ratio between the applied loads and the corresponding ultimate capacity or buckling strength.

2.2.2 For combined loads, the utilisation factor η_{act} is to be defined as the ratio of the applied equivalent stress and the corresponding buckling capacity, as shown in Fig 1, and is to be taken as:

$$\eta_{act} = \frac{W_{act}}{W_u} = \frac{1}{\gamma_c}$$

where:

W_{act} : Applied equivalent stress, in N/mm²:

- for plates:

$$W_{act} = \sqrt{\sigma_x^2 + \sigma_y^2 + \tau^2}$$

- for stiffeners:

$$W_{act} = \sigma_a + \sigma_b + \sigma_w$$

σ_x, σ_y : Membrane stresses, in N/mm², applied, respectively, in x direction and in y direction

τ : Membrane shear stress applied in xy plane, in N/mm²

σ_a : Actual stress in the stiffener, in N/mm², as defined in Sec 5, [2.3]

σ_b : Bending stress in the stiffener, in N/mm², as defined in Sec 5, [2.3]

σ_w : Warping stress in the stiffener, in N/mm², as defined in Sec 5, [2.3]

W_u : Equivalent buckling capacity, in N/mm², to be taken as:

- for plates:

$$W_u = \sqrt{\sigma_{cx}^2 + \sigma_{cy}^2 + \tau_c^2}$$

- for stiffeners:

$$W_u = \frac{R_{eH-S}}{S}$$

σ_{cx}, σ_{cy}, τ_c : Critical stresses, in N/mm², defined in Sec 5, [2.2] for plates and in Sec 5, [2.3] for stiffeners

R_{eH-S} : Specified minimum yield stress of the stiffener, in N/mm²

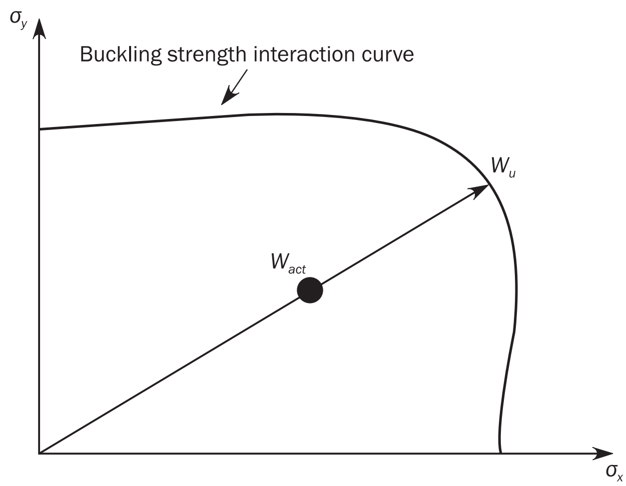
S : Partial safety factor, as defined in Sec 5

γ_c : Stress multiplier factor at failure.

For each typical failure mode, the corresponding capacity of the panel is calculated by applying the actual stress combination and then increasing or decreasing the stresses proportionally until collapse.

Fig 1 illustrates the buckling capacity and the buckling utilisation factor of a structural member subjected to σ_x and σ_y stresses.

Figure 1 : Example of buckling capacity and buckling utilisation factor



2.3 Allowable buckling utilisation factor

2.3.1 General structural elements

The allowable buckling utilisation factor η_{all} is defined in the applicable Rules.

2.4 Buckling acceptance criteria

2.4.1 A structural member is considered to have an acceptable buckling strength when it satisfies the following criterion:

$$\eta_{act} \le \eta_{all}$$

where:

η_{act} : Buckling utilisation factor based on the applied stress, defined in [2.2.2]

η_{all} : Allowable buckling utilisation factor as defined in [2.3].

SECTION 2 SLENDERNESS REQUIREMENTS

Symbols

$b_{f\text{-}out}$: Maximum distance, in mm, from mid-thickness of the web to the flange edge, as shown in Fig 1

b : Breadth of the unstiffened part of the plating between stiffeners and/or primary supporting members, in mm

ℓ : Span of stiffeners, in m

s : Stiffener spacing, in mm

h_w : Depth of stiffener web, in mm, as shown in Fig 1

R_{eH} : Specified minimum yield stress, in N/mm^2

ℓ_b : Effective length of bracket edge, in mm, as defined in Tab 3

s_{eff} : Effective width of stiffener attached plating, in mm, taken equal to:

$s_{eff} = 0,8$

t_f : Net thickness of stiffener flange, in mm

t_p : Net thickness of plate, in mm

t_w : Net thickness of stiffener web, in mm.

1 Structural elements

1.1 General

1.1.1 All the structural elements are to comply with the applicable slenderness and proportion requirements given in Articles [2] to [4].

2 Plates

2.1 Net thickness of plate panels

2.1.1 The net thickness of plate panels is to satisfy the following criterion:

$$t_p \geq \frac{b}{C}\sqrt{\frac{R_{eH}}{235}}$$

where:

C : Slenderness coefficient taken as:

- C = 100 for hull envelope
- C = 125 for the other structures.

R_{eH} : Specified minimum yield stress of the plate material, in N/mm^2.

A lower specified minimum yield stress may be used in this slenderness criterion provided the requirements specified in Sec 3 and Sec 4 are satisfied for the strake assumed in the same lower specified minimum yield stress value.

This requirement does not apply to the bilge plates within the cylindrical part of the ship and radius gunwale.

3 Stiffeners

3.1 Proportions of stiffeners

3.1.1 Net thickness of all stiffener types

The net thickness of stiffeners is to satisfy the following criteria:

a) Stiffener web plate:

$$t_w \geq \frac{h_w}{C_w}\sqrt{\frac{R_{eH}}{235}}$$

b) Stiffener flange:

$$t_f \geq \frac{b_{f\text{-}out}}{C_f}\sqrt{\frac{R_{eH}}{235}}$$

where:

C_w, C_f : Slenderness coefficients given in Tab 1.

3.1.2 Net dimensions of angle bars and T-bars

The total flange breadth b_f, in mm, for angle bars and T-bars is to satisfy the following criterion:

$b_f \geq 0,25\ h_w$

Figure 1 : Stiffener scantling parameters

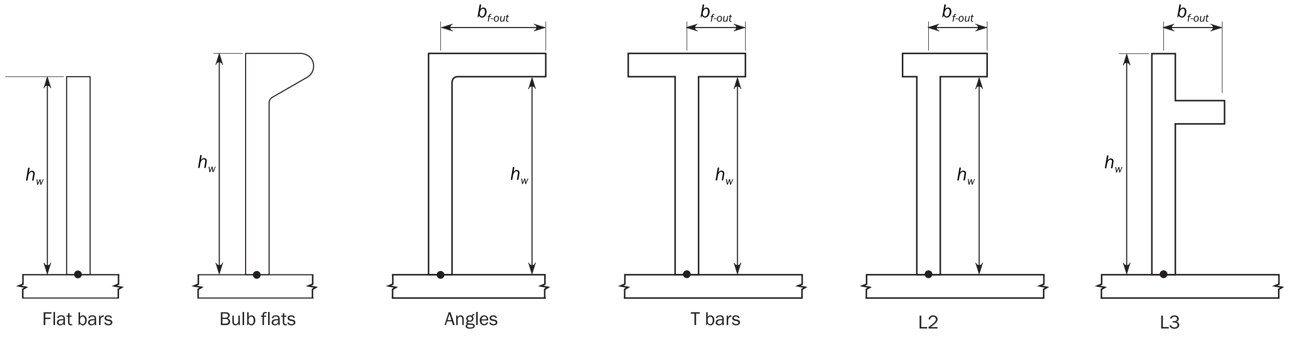


Table 1 : Slenderness coefficients C_w and C_f

Type of stiffeners	C_w	C_f
Angle, L2 and L3 bars	75	12
T-bars	75	12
Bulb bars	45	–
Flat bars	22	–

4 Primary supporting members

4.1 Proportions and stiffness

4.1.1 Proportions of web plates and flanges

The net thicknesses (web plate and flange) of primary supporting members are to satisfy the following criteria:

a) Web plates:

$$t_w \ge \frac{s_w}{C_w}\sqrt{\frac{R_{eH}}{235}}$$

b) Flanges:

$$t_f \ge \frac{b_{f\text{-}out}}{C_f}\sqrt{\frac{R_{eH}}{235}}$$

where:

s_w : Plate breadth, in mm, taken as the spacing of the web stiffeners

C_w : Slenderness coefficient for the web plates taken as $C_w = 100$

C_f : Slenderness coefficient for the flanges taken as: $C_f = 12$

4.1.2 Stiffness of deck transverse primary supporting members

The net moment of inertia $I_{psm\text{-}n50}$, in cm^4, of deck transverse primary members supporting deck longitudinals subject to axial compressive hull girder stress is to comply, within the central half of the bending span, with the following criterion:

$$I_{psm\text{-}n50} \ge 300\,\frac{\ell_{bdg}^4}{S^3 s}\,I_{st}$$

where:

$I_{psm\text{-}n50}$: Net moment of inertia, in cm^4, of deck transverse primary supporting members with an effective width of attached plating equal to 0,8 S

ℓ_{bdg} : Effective bending span of deck transverse primary supporting members, in m, as defined in the applicable Rules

S : Spacing of deck transverse primary supporting members, in m, as defined in the applicable Rules

I_{st} : Net moment of inertia of deck stiffeners, in cm^4, within the central half of the bending span, taken equal to:

$$I_{st} = 1,43\ \ell^2\ A_{eff}\ \frac{R_{eH}}{235}$$

A_{eff} : Net sectional area of the stiffener, including its effective attached plating s_{eff} , in cm^2

R_{eH} : Specified minimum yield stress of the material of the stiffener attached plating, in N/mm^2.

4.2 Web stiffeners fitted on primary supporting members

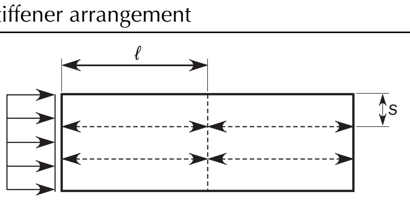
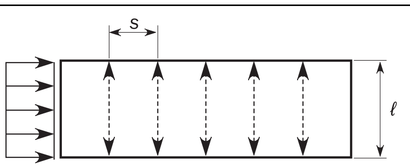
4.2.1 Proportions of web stiffeners

The net thicknesses (web plate and flange) and dimensions of the web stiffeners fitted on primary supporting members are to satisfy the requirements specified in [3.1.1] and [3.1.2].

4.2.2 Stiffness of web stiffeners

The net moment of inertia I_{st} , in cm^4, of web stiffeners fitted on primary supporting members, with effective attached plating s_{eff} , is not to be less than the minimum moment of inertia defined in Tab 2.

Table 2 : Stiffness criteria for web stiffeners fitted on primary supporting members (PSM)

Stiffener arrangement			Minimum moment of inertia of web stiffeners, in cm^4
A	Web stiffeners fitted along the PSM span		$I_{st} \ge 0,72\ \ell^2\ A_{eff}\ \frac{R_{eH}}{235}$
B	Web stiffeners fitted normal to the PSM span		$I_{st} \ge 1,14\ \ell\ s^2\ t_w\left(2,5\ \frac{1000\ell}{s} - 2\ \frac{s}{1000\ell}\right)\frac{R_{eH}}{235}\ 10^{-5}$

Note 1: ℓ : Length of the web stiffeners, in m:

- for web stiffeners welded to local supporting members, the length is to be measured between the flanges of the local support members
- for sniped web stiffeners, the length is to be measured between the lateral supports, i.e. corresponds to the total distance between the flanges of the primary supporting member, as shown for stiffener arrangement B

Note 2: A_{eff} : Net sectional area, in cm^2, of the web stiffener, including its effective attached plating s_{eff}

t_w : Net web thickness of the primary supporting member, in mm

R_{eH} : Specified minimum yield stress of the material of the web plate of the primary supporting member, in N/mm^2.

5 Brackets

5.1 Tripping brackets

5.1.1 Unsupported flange length

The unsupported length of the flange of the primary supporting members, in m, i.e. the distance between tripping brackets, is to satisfy the following criterion:

$$S_b \le \mathrm{Max}\left(b_f \ C \sqrt{\frac{A_{f\text{-}n50}}{\left(A_{f\text{-}n50} + \dfrac{A_{w\text{-}n50}}{3}\right)} \left(\frac{235}{R_{eH}}\right)} \ ; \ S_{b\text{-}min} \right)$$

where:

b_f : Flange breadth of primary supporting members, in mm

C : Slenderness coefficient taken as:

- $C = 0{,}022$ for symmetrical flanges
- $C = 0{,}033$ for asymmetrical flanges

$A_{f\text{-}n50}$: Net cross-sectional area of the flange, in cm²

$A_{w\text{-}n50}$: Net cross-sectional area of the web plate, in cm²

R_{eH} : Specified minimum yield stress of the PSM material, in N/mm²

$S_{b\text{-}min}$: Minimum unsupported flange length, in m, taken as:

- $S_{b\text{-}min} = 3{,}0$ m for cargo tank/hold region, on tank/hold boundaries or on hull envelope including external decks
- $S_{b\text{-}min} = 4{,}0$ m for the other areas.

5.1.2 Edge stiffening

The tripping brackets on primary supporting members are to be stiffened by a flange or an edge stiffener if the effective length of the edge ℓ_b, as defined in Tab 3, in mm, is greater than $75\ t_b$, where:

t_b : Net web thickness of the brackets, in mm.

5.2 End brackets

5.2.1 Proportions

The net web thickness, in mm, of the end brackets subjected to compressive stresses is to satisfy the following criterion:

$$t_b \ge \frac{d_b}{C} \sqrt{\frac{R_{eH}}{235}}$$

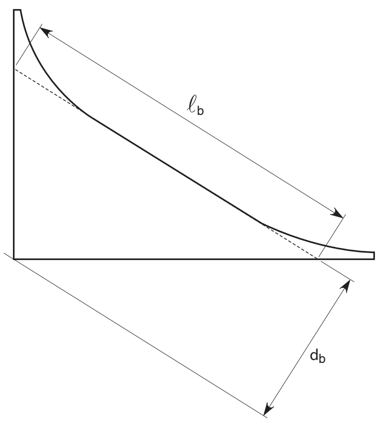
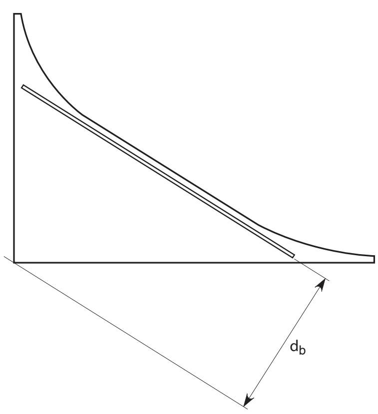
where:

d_b : Bracket depth, in mm, as defined in Tab 3

C : Slenderness coefficient as defined in Tab 3

R_{eH} : Specified minimum yield stress of the end bracket material, in N/mm².

Table 3 : Slenderness coefficient C for proportions of brackets

Mode	C
Brackets without edge stiffener 	$C = 20\left(\frac{d_b}{\ell_b}\right) + 16$ with: $0,25 \le \frac{d_b}{\ell_b} \le 1,0$
Brackets with edge stiffener 	$C = 70$

5.3 Edge reinforcement

5.3.1 Reinforcement of bracket edges

The web depth h_w, in mm, of the edge stiffeners in way of brackets is to satisfy the following criterion:

$$h_w \ge \mathrm{Max}\left(C \ell_b \sqrt{\frac{R_{eH}}{235}} \cdot 10^{-3} ; 50\right)$$

where:

C : Slenderness coefficient, taken as:

- $C = 75$ for end brackets
- $C = 50$ for tripping brackets

R_{eH} : Specified minimum yield stress of the stiffener material, in N/mm².

5.3.2 Proportions of edge stiffeners

The net thicknesses (web plate and flange) and dimensions of the edge stiffeners are to satisfy the requirements specified in [3.1.1] and [3.1.2].

6 Other structures

6.1 Pillars

6.1.1 Proportions of I-section pillars

The net thicknesses (web plate and flanges) and dimensions of I-section pillars are to comply with the requirements specified in [3.1.1] and [3.1.2].

6.1.2 Proportions of box section pillars

The net thickness of thin-walled box section pillars is to comply with the requirements specified in [3.1.1], item a).

6.1.3 Proportions of circular section pillars

The net thickness t, in mm, of circular section pillars is to comply with the following criterion:

$$t \geq \frac{r}{50}$$

where:

r : Mid-thickness radius of the circular section, in mm.

6.2 Edge reinforcement in way of openings

6.2.1 Depth of edge stiffeners

When fitted as shown in Fig 2, the web depth h_w, in mm, of edge stiffeners in way of openings is to satisfy the following criterion:

$$h_w \geq \mathrm{Max}\left(C\ell\sqrt{\frac{R_{eH}}{235}}\ ;\ 50\right)$$

where:

C : Slenderness coefficient taken as:

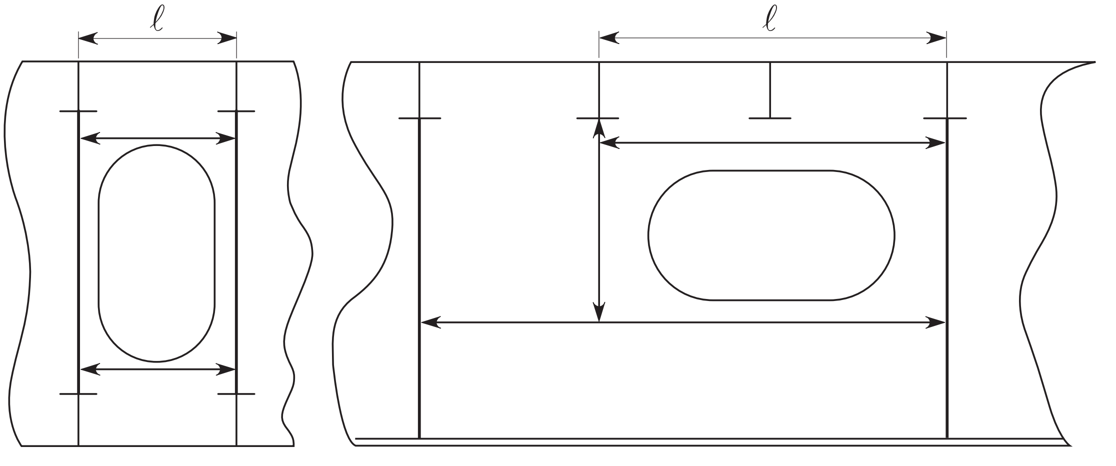
C = 50

R_{eH} : Specified minimum yield stress of the edge stiffener material, in N/mm².

6.2.2 Proportions of edge stiffeners

The net thicknesses (web plate and flange) and dimensions of the edge stiffeners are to satisfy the requirements specified in [3.1.1] and [3.1.2].

Figure 2 : Typical edge reinforcements



SECTION 3 PRESCRIPTIVE BUCKLING REQUIREMENTS

Symbols

η_{all} : Allowable buckling utilisation factor, as defined in Sec 1, [2.3]

EPP : Elementary Plate Panel, i.e. the unstiffened part of the plating between stiffeners and/or primary supporting members

LCP : Load Calculation Point, as defined in the applicable Rules.

1 General

1.1 Scope

1.1.1 This Section applies to plate panels, including curved plate panels, and stiffeners subject to hull girder compression and shear stresses. In addition the following structural members subject to compressive stresses are to be checked:

- corrugations of transverse vertically corrugated bulkheads
- corrugations of longitudinal corrugated bulkheads
- struts
- pillars
- cross ties.

1.1.2 The hull girder buckling strength requirements apply along the full length of the ship.

1.1.3 Design load sets

The buckling checks are to be performed for all design load sets, with pressure combination defined in the applicable Rules.

For each design load set, and for all dynamic load cases, the lateral pressure is to be determined and applied at a load calculation point as described in the applicable Rules. It is to be applied together with the hull girder stress combinations given in [2.1].

1.2 Equivalent plate panel

1.2.1 In longitudinal stiffening arrangement, when the plate thickness varies over the width b of a plate panel, the buckling check is to be performed for an equivalent plate panel width, combined with the smaller plate thickness t_1. The width b_{eq} of this equivalent plate panel, in mm, is defined by the following formula:

$$b_{eq} = \ell_1 + \ell_2 \left(\frac{t_1}{t_2}\right)^{1,5}$$

where:

ℓ_1 : Width of the part of the plate panel with the smaller net plate thickness t_1, in mm, as defined in Fig 1

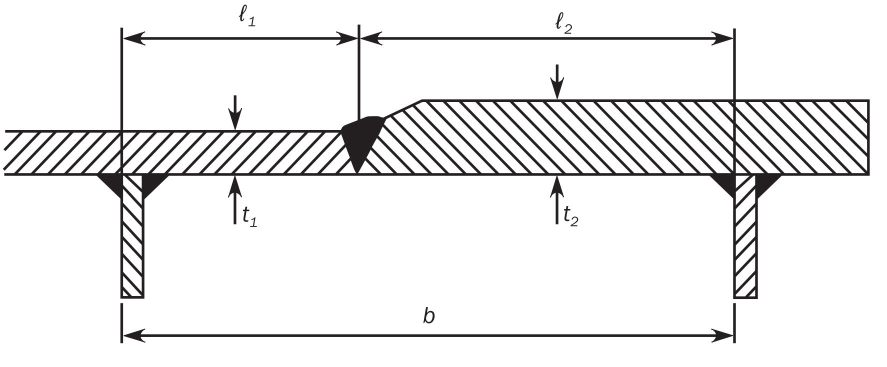
ℓ_2 : Width of the part of the plate panel with the greater net plate thickness t_2, in mm, as defined in Fig 1.

1.2.2 In transverse stiffening arrangement, when an EPP is made with different thicknesses, the buckling check of the plate and stiffeners is to be made for each thickness considered constant on the EPP, the stresses and pressures being estimated for the EPP at the LCP.

1.2.3 Materials

When the plate panel is made of different materials, the minimum yield strength is to be used for the buckling assessment.

Figure 1 : Plate thickness change over the width b



2 Hull girder stress

2.1 General

2.1.1 Each elementary plate panel and each stiffener are to satisfy the criteria defined in [3] with the following stress combinations:

a) Longitudinal stiffening arrangement:

$\sigma_x = \sigma_{hg}$

$\sigma_y = 0$

$\tau = \tau_{hg}$

b) Transverse stiffening arrangement:

$\sigma_x = 0$

$\sigma_y = \sigma_{hg}$

$\tau = \tau_{hg}$

where:

σ_{hg} : Hull girder bending stress, in N/mm², in the elementary plate panel or in the stiffener, determined according to the applicable Rules

τ_{hg} : Hull girder shear stress, in N/mm², in the elementary plate panel or in the stiffener attached plating, determined according to the applicable Rules.

3 Buckling criteria

3.1 Overall stiffened panels

3.1.1 The buckling strength of overall stiffened panels is to satisfy the following criterion:

$\eta_{Overall} \leq \eta_{all}$

where:

$\eta_{Overall}$: Maximum utilisation factor, as defined in Sec 5, [2.1].

3.2 Elementary plate panels

3.2.1 The buckling strength of elementary plate panels is to satisfy the following criterion:

$\eta_{Plate} \leq \eta_{all}$

where:

η_{Plate} : Maximum plate utilisation factor calculated according to SP-A, as defined in Sec 5, [2.2].

For the determination of η_{Plate} of the vertically stiffened side shell plating of single side skin ships between hopper and topside tanks, the cases 12 and 16 of Sec 5, Tab 4 corresponding to the shorter edge of the plate panel clamped are to be considered together with a mean σ_y stress and Ψ_y=1.

3.3 Stiffeners and side frames of single-side skin ships

3.3.1 The buckling strength of stiffeners or of side frames of single-side skin ships between hopper and topside tanks is to satisfy the following criterion:

$\eta Stiffener \leq \eta all$

where:

$\eta_{Stiffener}$: Maximum stiffener utilisation factor, as defined in Sec 5, [2.3].

Note 1: This criterion check can only be fulfilled when the overall stiffened panel criterion, as defined in [3.1.1], is satisfied.

3.4 Vertically corrugated transverse and longitudinal bulkheads

3.4.1 The shear buckling strength of vertically corrugated transverse and longitudinal bulkheads is to satisfy the following criterion:

$\eta_{Shear} \leq \eta_{all}$

where:

η_{Shear} : Maximum shear corrugated bulkhead utilisation factor:

$$h_{Shear} = \frac{\tau_{bhd}}{\tau_c}$$

τ_{bhd} : Shear stress, in N/mm², in the bulkhead taken as:

- for longitudinal bulkheads: the hull girder shear stress defined in Article [2]
- for transverse bulkheads: the shear stress in the corrugation defined in the applicable Rules

τ_c : Shear critical stress, in N/mm², as defined in Sec 5, [2.2.3].

3.5 Horizontally corrugated longitudinal bulkheads

3.5.1 Each corrugation, within the extension of half flange, web and half flange, is to satisfy the following criterion:

$\eta \leq \eta_{all}$

where:

η : Overall column utilisation factor, as defined in Sec 5, [3.1].

3.6 Struts, pillars and cross ties

3.6.1 The compressive buckling strength of struts, pillars and cross ties is to satisfy the following criterion:

$\eta \leq \eta_{all}$

where:

η : Maximum buckling utilisation factor of struts, pillars or cross ties, as defined in Sec 5, [3.1].

SECTION 4 BUCKLING REQUIREMENTS FOR DIRECT STRENGTH ANALYSIS

Symbols

η_{all} : Allowable buckling utilisation factor, as defined in Sec 1, [2.3]

α : Aspect ratio of the plate panel, defined in Sec 5.

1 General

1.1 Scope

1.1.1 The requirements of this Section apply for the buckling assessment of direct strength analysis subjected to compressive stress, shear stress and lateral pressure.

1.1.2 All structural elements in the FE analysis are to be assessed individually. The buckling checks are to be performed for the following structural elements:

- stiffened and unstiffened panels, including curved panels
- web plates in way of openings
- corrugated bulkheads
- vertically stiffened side shell, between hopper and topside tanks, of single-side skin ships
- struts, pillars and cross ties.

2 Stiffened and unstiffened panels

2.1 General

2.1.1 The plate panels of hull structure are to be modelled as stiffened or unstiffened panels. Method A or Method B as defined in Sec 1, [2] is to be used according to App 2.

2.1.2 Average thickness of plate panel

Where the plate thickness along a plate panel is not constant, the panel used for the buckling assessment is to be modelled according to the applicable Rules, with a weighted average thickness t_{avr}, in mm, taken as:

$$t_{avr} = \frac{\sum_{1}^{n} A_i t_i}{\sum_{1}^{n} A_i}$$

where:

A_i : Area of the i-th plate element, in mm^2

t_i : Net thickness of the i-th plate element, in mm

n : Number of finite elements defining the buckling plate panel.

2.1.3 Yield stress of plate panel

The yield stress R_{eH_P}, in N/mm^2, of a plate panel is taken as the minimum value of the specified yield stresses of the elements within the plate panel.

2.2 Stiffened panels

2.2.1 To represent the overall buckling behaviour, each stiffener with attached plating is to be modelled as a stiffened panel of the extent defined in App 2, Tab 1.

2.2.2 If the stiffener properties or the stiffener spacing vary within the stiffened panel, the calculations are to be performed separately for all the configurations of the plate panels, i.e. for each stiffener and plate between the stiffeners. The plate thickness, stiffener properties and stiffener spacing at the considered location are to be assumed for the whole panel.

2.3 Unstiffened panels

2.3.1 Irregular panel

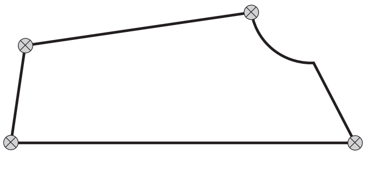
In way of web frames, stringers and brackets, the geometry of the panel (i.e. plate bounded by web stiffeners/face plates) may not have a rectangular shape. In this case, an equivalent rectangular panel is to be defined according to [2.3.2] for irregular geometry and [2.3.3] for triangular geometry and is to comply with buckling assessment.

2.3.2 Modelling of an unstiffened panel with irregular geometry

Unstiffened panels with irregular geometry are to be idealised to equivalent rectangular panels for plate buckling assessment according to the following procedure:

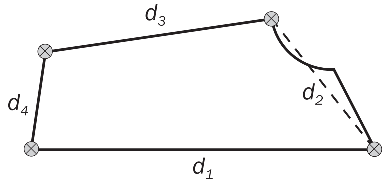
a) the four corners closest to a right angle (90 deg) in the bounding polygon for the plate are identified as shown in Fig 1.

Figure 1 : Unstiffened panel



b) the distances along the plate bounding polygon between the corners (as shown in Fig 2) are calculated, i.e. the sum of all the straight line segments between the end points.

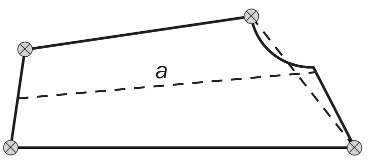
Figure 2 : Distances



c) the pair of opposite edges with the smallest total length is identified, i.e. the minimum of ($d_1 + d_3$) and ($d_2 + d_4$)

d) a line joins the middle points of the chosen opposite edges as shown in Fig 3 (a middle point is defined as the point at half the distance from one end). This line defines the longitudinal direction for the capacity model. The length of the line defines the length a of the capacity model, measured from one end point.

Figure 3 : Join line a

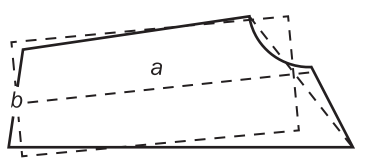


e) the length b of the shorter side, in mm, as shown in Fig 4, is to be taken as:

$b = A/a$

where:

Figure 4 : Side b



A : Area of the plate, in mm²

a : Length, in mm, defined in item d).

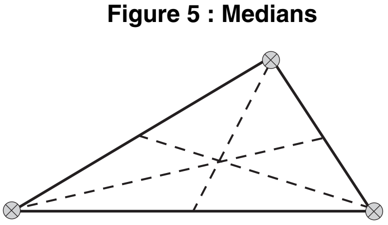
f) the stresses from the direct strength analysis are to be transformed into the local coordinate system of the equivalent rectangular panel. These stresses are to be used for the buckling assessment.

2.3.3 Modelling of an unstiffened panel with triangular geometry

Unstiffened panels with triangular geometry are to be idealised to equivalent rectangular panels for plate buckling assessment according to the following procedure:

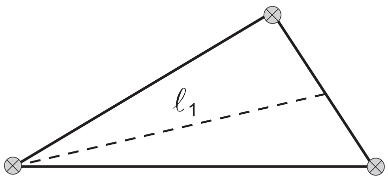
a) medians are constructed as shown in Fig 5.

Figure 5 : Medians



b) the longest median is identified as shown in Fig 6. This median, the length of which is ℓ_1 in mm, defines the longitudinal direction for the capacity model.

Figure 6 : Longest median ℓ_1



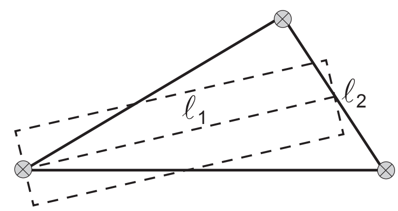
c) the width ℓ_2 of the model, in mm, as shown in Fig 7, is to be taken as:

$\ell_2 = A/\ell_1$

where:

A : Area of the plate, in mm².

Figure 7 : Width ℓ_2 of the model



d) the lengths of the shorter side b and the longer side a, in mm, of the equivalent rectangular panel are to be taken as:

$$b = \frac{\ell_2}{C_{tri}}$$

$$a = \ell_1 \, C_{tri}$$

where:

$$C_{tri} = 0,4\frac{\ell_2}{\ell_1} + 0,6$$

e) the stresses from the direct strength analysis are to be transformed into the local coordinate system of the equivalent rectangular panel. These stresses are to be used for the buckling assessment.

2.4 Reference stresses

2.4.1 The stress distribution is to be taken from the direct strength analysis and applied to the buckling model.

2.4.2 The reference stresses are to be calculated using the stress based reference stresses, as defined in App 1.

2.5 Lateral pressure

2.5.1 The lateral pressure applied to the direct strength analysis is also to be applied to the buckling assessment unless otherwise stated in the applicable Rules.

2.5.2 Where the lateral pressure is not constant over a buckling panel defined by a number of finite plate elements, an average lateral pressure P_{avr}, in N/mm², is calculated using the following formula:

$$P_{avr} = \frac{\sum_1^n A_i P_i}{\sum_1^n A_i}$$

where:

- A_i : Area of the i-th plate element, in mm²
- P_i : Lateral pressure of the i-th plate element, in N/mm²
- n : Number of finite elements in the buckling panel.

2.6 Buckling criteria

2.6.1 UP-A

The compressive buckling strength of UP-A is to satisfy the following criterion:

$\eta_{UP-A} \leq \eta_{all}$

where:

η_{UP-A} : Maximum plate panel utilisation factor calculated according to Method A, as defined in Sec 5, [2.2].

2.6.2 UP-B

The compressive buckling strength of UP-B is to satisfy the following criterion:

$\eta_{UP-B} \leq \eta_{all}$

where:

η_{UP-B} : Maximum plate panel utilisation factor calculated according to Method B, as defined in Sec 5, [2.2].

2.6.3 SP-A

The compressive buckling strength of SP-A is to satisfy the following criterion:

$\eta_{SP-A} \leq \eta_{all}$

where:

η_{SP-A} : Maximum stiffened panel utilisation factor taken as the maximum of:

- the overall stiffened panel capacity, as defined in Sec 5, [2.1]
- the plate capacity calculated according to Method A, as defined in Sec 5, [2.2]
- the stiffener buckling strength as defined in Sec 5, [2.3], considering separately the properties (thickness, dimensions), the pressures defined in [2.5.2] and the reference stresses of each EPP at both sides of the stiffener.

Note 1: The stiffener buckling capacity check can only be fulfilled when the overall stiffened panel capacity, as defined in Sec 5, [2.1], is satisfied.

2.6.4 SP-B

The compressive buckling strength of SP-B is to satisfy the following criterion:

$\eta_{SP-B} \leq \eta_{all}$

where:

η_{SP-B} : Maximum stiffened panel utilisation factor taken as the maximum of:

- the overall stiffened panel capacity, as defined in Sec 5, [2.1]
- the plate capacity calculated according to Method B, as defined in Sec 5, [2.2]
- the stiffener buckling strength as defined in Sec 5, [2.3], considering separately the properties (thickness, dimensions), the pressures defined in [2.5.2] and the reference stresses of each EPP at both sides of the stiffener.

Note 1: The stiffener buckling capacity check can only be fulfilled when the overall stiffened panel capacity, as defined in Sec 5, [2.1], is satisfied.

2.6.5 Web plate in way of openings

The web plate of primary supporting members in way of openings is to satisfy the following criterion:

$\eta_{opening} \leq \eta_{all}$

where:

$\eta_{opening}$: Maximum web plate utilisation factor in way of openings, as defined in Sec 5, [2.4].

3 Corrugated bulkheads

3.1 General

3.1.1 Three buckling failure modes are to be assessed on corrugated bulkheads:

- corrugation overall column buckling
- corrugation flange panel buckling
- corrugation web panel buckling.

3.2 Reference stresses

3.2.1 Each corrugation flange and web panel is to be assessed.

3.2.2 The membrane stresses at element centroid are to be used.

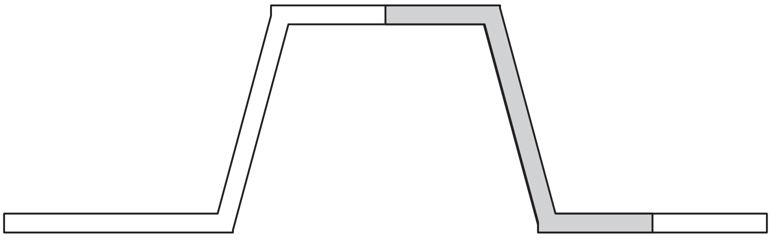
3.2.3 For the application of this requirement:

b : Width of the considered member (flange or web) of the corrugation.

The maximum normal stress σ_x parallel to the corrugation is the maximum of the two following stresses:

- the normal stress parallel to the corrugation taken at b/2 from the corrugation ends
- the normal stress parallel to the corrugation within the mid-span of the corrugation.

Figure 8 : Single corrugation



When a corrugation end is fitted with a shedder plate, the normal stress parallel to the corrugation at this end is to be taken at b/2 from the intersection of the shedder plate with the point at mid-breadth of the flange or of the web, as the case may be.

The maximum shear stress is the shear stress which is maximum at the corrugation flange or web at the point b/2 from ends as defined above for the normal stress parallel to the corrugation.

The in-plane stresses σ_x and σ_y and the shear stress τ are to be taken as the element stresses averaged over the width of the considered member (flange or web) at the considered location.

When the stress value at b/2 from ends cannot be obtained directly from FEA element, the stress at this location is to be obtained by interpolation. This interpolation is to be made on elements extending over a distance equal to 3 b at a point located at b/2 from the end of the corrugation or from the intersection of the shedder plate if fitted, measured at mid-breadth of the flange or of the web. The interpolation of the in-plane stresses σ_x and σ_y is to be made in accordance with App 1, [2.1].

The shear stress at b/2 is obtained by linear interpolation between the elements the closest to b/2 location.

3.2.4 Where more than one plate thickness is used for a flange panel, the maximum stress is to be obtained for each thickness range and is to be checked with the buckling criteria for each thickness.

3.3 Overall column buckling

3.3.1 The overall buckling failure mode of corrugated bulkheads subjected to axial compression is to be checked for column buckling (e.g. horizontally corrugated bulkheads and vertically corrugated bulkheads subjected to local vertical forces).

Table 1 : Application of overall column buckling for corrugated bulkheads

Bulkhead orientation	Corrugation orientation	
	Horizontal	Vertical
Longitudinal or Transverse	Required	Required, when subjected to local vertical forces (e.g. crane loads)

3.3.2 Each corrugation unit, i.e. each single corrugation made up of half flange/web/half flange, as shown in grey in Fig 8, is to satisfy the following criterion:

$\eta_{Overall} \leq \eta_{all}$

where:

$\eta_{Overall}$: Maximum overall column utilisation factor, as defined in Sec 5, [3.1] and Sec 5, [3.1.2], considering the corrugation unit as a pillar with an unsupported length equal to the length of the corrugation.

3.3.3 End constraint factor f_{end} to be applied corresponds to:

- pinned ends, in general
- fixed end support, in case of stool having a width exceeding 2 times the depth of the corrugation.

3.4 Local buckling

3.4.1 The compressive buckling strength of a unit flange and a unit web of corrugated bulkheads is to satisfy the following criterion:

$\eta_{Corr} \leq \eta_{all}$

where:

η_{Corr} : Maximum unit flange or unit web utilisation factor, as defined in Sec 5, [3.2.1].

Two stress combinations are to be considered for the application of this criterion:

- the maximum normal stress σ_x parallel to the corrugation, combined with the stress σ_y perpendicular to the corrugation and with the shear stress τ, at the location where the maximum normal stress parallel to the corrugation occurs.
- the maximum shear stress τ, combined with the normal stress σ_x parallel to the corrugation and with the stress σ_y perpendicular to the corrugation, at the location where the maximum shear stress occurs.

The buckling assessment is to be performed with an aspect ratio α equal to 2, and for the member thicknesses where the maximum compressive/shear stress occurs (see [3.2.4]).

4 Vertically stiffened side shell of single-side skin ships

4.1 Buckling criteria

4.1.1 Side shell plating

The compressive buckling strength of the vertically stiffened side shell plating of single-side skin ships, between hopper and topside tanks, is to satisfy the following criterion:

$\eta_{VSS} \leq \eta_{all}$

where:

η_{VSS} : Maximum vertically stiffened side shell plating utilisation factor calculated according to Method A as defined in Sec 5, [2.2.1] and considering the boundary conditions and stress combinations detailed in a) and b) hereafter:

a) 4 edges simply supported (cases 1, 2 and 15 of Sec 5, Tab 4):

- Pure vertical stress:

  The maximum vertical stress of stress elements is used with $\alpha = 1$ and $\Psi_x = 1$.

- Maximum vertical stress combined with longitudinal and shear stress:

  The maximum vertical stress in the buckling panel plus the shear and longitudinal stresses at the location where the maximum vertical stress occurs is used with $\alpha = 2$ and $\Psi_x = \Psi_y = 1$

  The plate thickness to be considered in the buckling strength check is the one where the maximum vertical stress occurs.

- Maximum shear stress combined with longitudinal and vertical stress:

  The maximum shear stress in the buckling panel plus the longitudinal and vertical stresses at the location where maximum shear stress occurs is used with $\alpha = 2$ and $\Psi_x = \Psi_y = 1$

  The plate thickness to be considered in the buckling strength check is the one where the maximum shear stress occurs.

b) The 2 shorter edges of the plate panel clamped (cases 11, 12 and 16 of Sec 5, Tab 4):

- Distributed longitudinal stress associated with vertical and shear stress:

  The actual size of the buckling panel is used to define α

  The average values for longitudinal, vertical and shear stresses are to be used

  $\Psi_x = \Psi_y = 1$

  The plate thickness to be considered in the buckling strength check is the minimum thickness of the buckling panel.

4.1.2 Side frames

The buckling strength of side frames of single-side skin ships, between hopper and topside tanks, is to satisfy the following criterion:

$\eta Stiffener \leq \eta all$

where:

$\eta_{Stiffener}$: Maximum stiffener utilisation factor, as defined in Sec 5, [2.3].

5 Struts, pillars and cross ties

5.1 Buckling criteria

5.1.1 The compressive buckling strength of struts, pillars and cross ties is to satisfy the following criterion:

$\eta_{Pillar} \leq \eta_{all}$

where:

η_{Pillar} : Maximum utilisation factor of struts, pillars or cross ties, as defined in Sec 5, [3.1].

SECTION 5 BUCKLING CAPACITY

Symbols

A_s : Net sectional area of the stiffener without attached plating, in mm²

a : Length of the longer side of the plate panel, in mm

b : Length of the shorter side of the plate panel, in mm

b_{eff} : Effective width of the attached plating of a stiffener, in mm, as defined in [2.3.5]

b_{eff1} : Effective width of the attached plating of a stiffener, in mm, without the shear lag effect, taken as:

- when $\sigma_x > 0$
  - for prescriptive assessment:

    $$b_{eff1} = \frac{C_{x1} b_1 + C_{x2} b_2}{2}$$

  - for FE analysis:

    $b_{eff1} = C_x\, b$
- when $\sigma_x \le 0$

  $b_{eff1} = b$

b_f : Breadth of the stiffener flange, in mm

b_1, b_2 : Width of the plate panel on each side of the considered stiffener, in mm

C_{x1}, C_{x2} : Tab 4, calculated for the EPP1 and EPP2 on each side of the considered stiffener according to case 1

C_x : Reduction factor as defined in [2.2.3]

d : Length, in mm, of the side parallel to the axis of the cylinder corresponding to the curved plate panel, as shown in Tab 5

d_e : Distance, in mm, from the upper edge of web to the top of the flange, as shown in Fig 1

E : Young's modulus of the material, in N/mm²

e_f : Distance, in mm, from the attached plating to the flange centre, as shown in Fig 1, depending on the profile type:

- $e_f = h_w$ for flat bars
- $e_f = h_w - 0,5\ t_f$ for bulb bars
- $e_f = h_w + 0,5\ t_f$ for angle bars, T-bars and L2 bars
- $e_f = h_w - d_e - 0,5\ t_f$ for L3 bars.

F_{long} : Correction factor defined in [2.2.4]

F_{tran} : Correction factor defined in [2.2.5]

h_w : Depth of the stiffener web, in mm, as shown in Fig 1

ℓ : Span of the stiffener, in mm, equal to the spacing between the primary supporting members

R : Radius of the curved plate panel, in mm

R_{eH_P} : Specified minimum yield stress of the plate, in N/mm²

R_{eH_S} : Specified minimum yield stress of the stiffener, in N/mm²

S : Partial safety factor, to be taken as:

- for structures exposed to local concentrated loads:

  S = 1,1
- for stiffeners located on the hatchway coamings, the sloping plate of the topside and hopper tanks, the inner bottom, the inner side if any, the side shell of single-side skin construction between hopper and topside tanks and the transverse bulkheads top and bottom stools of ships assigned with the service notation **bulk carrier** and having a length greater than 150 m:

  S = 1,15
- for all the other cases:

  S = 1,0

t_p : Net thickness of the plate panel, in mm

t_w : Net thickness of the stiffener web, in mm

t_f : Net thickness of the stiffener flange, in mm

x axis : For a rectangular buckling panel, local axis parallel to its long edge

y axis : For a rectangular buckling panel, local axis perpendicular to its long edge

α : Aspect ratio of the plate panel, to be taken as:

$$\alpha = \frac{a}{b}$$

β : Coefficient taken as:

$$\beta = \frac{1-\psi}{\alpha}$$

ω : Coefficient taken as:

$\omega = \mathrm{Min}\,(3\ ;\ \alpha)$

σ_x : Stress applied on the edge along x axis of the buckling panel, in N/mm²

σ_y : Stress applied on the edge along y axis of the buckling panel, in N/mm²

σ_1 : Maximum stress, in N/mm²

σ_2 : Minimum stress, in N/mm²

σ_E : Elastic buckling reference stress, in N/mm², to be taken as:

- for the application of plate limit state according to [2.2.1]:

$$\sigma_E = \frac{\pi^2 E}{12(1-\nu^2)}\left(\frac{t_p}{b}\right)^2$$

- for the application of curved plate panels according to [2.2.5]:

$$\sigma_E = \frac{\pi^2 E}{12(1-\nu^2)}\left(\frac{t_p}{d}\right)^2$$

τ : Applied shear stress, in N/mm²

ν : Poisson's ratio of the material

ψ : Edge stress ratio, to be taken as:

$$\psi = \frac{\sigma_2}{\sigma_1}$$

γ : Stress multiplier factor acting on loads. When γ is such that the loads reach the interaction formulae, then:

$$\gamma = \gamma_c$$

γ_c : Stress multiplier factor at failure.

1 General

1.1 Scope

1.1.1 This Section contains the methods for determination of the buckling capacity of plate panels, stiffeners, primary supporting members, struts, pillars, cross ties and corrugated bulkheads.

1.1.2 For the application of this Section, the stresses σ_x , σ_y and τ applied on the structural members are defined in:

- Sec 3 for the prescriptive requirements
- Sec 4 for the FE analysis requirements.

1.1.3 Ultimate buckling capacity

The ultimate buckling capacity is calculated by applying the actual stress combination and then increasing or decreasing the stresses proportionally until the interaction formulae defined in [2.1.1], [2.2.1], and [2.3.4] are equal to 1,0.

1.1.4 Buckling utilisation factor

The buckling utilisation factor η of the structural member is equal to the highest utilisation factor obtained for the different buckling modes.

1.1.5 Lateral pressure

The lateral pressure is to be considered as constant in the buckling strength assessment.

2 Buckling capacity of plates and stiffeners

2.1 Overall stiffened panel capacity

2.1.1 The elastic stiffened panel limit state is based on the following interaction formula:

$$\frac{P_z}{c_f} = 1$$

where c_f and P_z are defined in [2.3.4].

2.2 Plate capacity

2.2.1 Plate limit state

a) The plate limit state is based on the following interaction formulae:

- $$\left(\frac{\gamma_{c1}\ \sigma_x\ S}{\sigma'_{cx}}\right)^{e_0} + \left(\frac{\gamma_{c1}\ \sigma_y\ S}{\sigma'_{cy}}\right)^{e_0} + \left(\frac{\gamma_{c1}\ |\tau|\ S}{\tau'_c}\right)^{e_0} - \Omega = 1$$

  with:

  $$\Omega = B\left(\frac{\gamma_{c1}\ \sigma_x\ S}{\sigma'_{cx}}\right)^{e_0/2}\left(\frac{\gamma_{c1}\ \sigma_y\ S}{\sigma'_{cy}}\right)^{e_0/2}$$

- when $\sigma_x \geq 0$:

  $$\left(\frac{\gamma_{c2}\ \sigma_x\ S}{\sigma'_{cx}}\right)^{2/\beta_p^{0,25}} + \left(\frac{\gamma_{c2}\ |\tau|\ S}{\tau'_c}\right)^{2/\beta_p^{0,25}} = 1$$

- when $\sigma_y \geq 0$:

  $$\left(\frac{\gamma_{c3}\ \sigma_y\ S}{\sigma'_{cy}}\right)^{2/\beta_p^{0,25}} + \left(\frac{\gamma_{c3}\ |\tau|\ S}{\tau'_c}\right)^{2/\beta_p^{0,25}} = 1$$

- $$\frac{\gamma_{c4}\ |\tau|\ S}{\tau'_c} = 1$$

where:

σ_x , σ_y : Normal stresses applied on the plate panel, in N/mm², to be taken as defined in [2.2.6]

τ : Shear stress applied on the plate panel, in N/mm²

σ_{cx}' : Ultimate buckling stress, in N/mm², in the direction parallel to the longer edge of the buckling panel, as defined in [2.2.3]

σ_{cy}' : Ultimate buckling stress, in N/mm², in the direction parallel to the shorter edge of the buckling panel, as defined in [2.2.3]

τ_c' : Ultimate buckling shear stresses, in N/mm², as defined in [2.2.3]

γ_{c1} , γ_{c2} , γ_{c3} , γ_{c4} : Stress multiplier factors at failure for each of the above different limit states.

γ_{c2} and γ_{c3} are to be considered only when $\sigma_x \geq 0$ and $\sigma_y \geq 0$, respectively

B, e_0 : Coefficients given in Tab 1.

b) The stress multiplier factor at failure, γ_c , is taken as:

$$\gamma_c = \text{Min}\ (\gamma_{c1}\ ;\ \gamma_{c2}\ ;\ \gamma_{c3}\ ;\ \gamma_{c4})$$

Figure 1 : Stiffener cross-sections

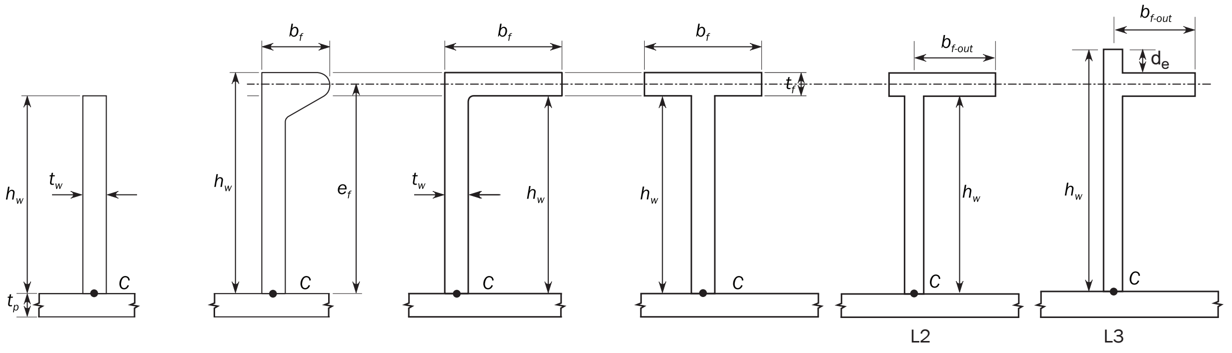


Table 1 : Coefficients B and e_0

Applied stresses	B	e_0
$\sigma_x \geq 0$ and $\sigma_y \geq 0$	$0{,}7 - 0{,}3\ \beta_p / \alpha^2$	$2/\beta_p^{0,25}$
$\sigma_x < 0$ or $\sigma_y < 0$	1,0	2,0
Note 1: β_p : Plate slenderness parameter taken as: $\beta_p = \frac{b}{t_p}\sqrt{\frac{R_{eH\_P}}{E}}$		

2.2.2 Reference degree of slenderness

The reference degree of slenderness is to be taken as:

$$\lambda = \sqrt{\frac{R_{eH\_P}}{K\sigma_E}}$$

where:

K : Buckling factor, as defined in Tab 4 for plane plate panels and Tab 5 for curved plate panels.

2.2.3 Ultimate buckling stresses

The ultimate buckling stresses of plate panels, in N/mm², are to be taken as:

$$\sigma_{cx}' = C_x\ R_{eH\_P}$$

$$\sigma_{cy}' = C_y\ R_{eH\_P}$$

The ultimate buckling stress of plate panels subject to shear, in N/mm², is to be taken as:

$$\tau_c' = C_\tau \frac{R_{eH\_P}}{\sqrt{3}}$$

where:

C_x , C_y , C_τ : Reduction factors, as defined in Tab 4.

- for the first equation of [2.2.1]:

  when $\sigma_x < 0$ or $\sigma_y < 0$, the reduction factors are to be taken as follows:

  $C_x = C_y = C_\tau = 1$
- in the other cases:

  C_y is calculated according to Tab 4, using the values of c_1 given in Tab 2.

The boundary conditions for the plates are to be considered as simply supported: see case 1, case 2 and case 15 of Tab 4.

If the boundary conditions differ significantly from the condition 'simple support', a more appropriate boundary condition can be applied, chosen from the different cases of Tab 4, subject to the agreement of the Society.

Table 2 : Coefficient c_1

Plate panels	c_1
SP-A	$c_1 = \left(1 - \frac{1}{\alpha}\right) \geq 0$
UP-A	
Vertically stiffened single-side skin between hopper and topside tanks	
Corrugations of corrugated bulkhead	
SP-B	$c_1 = 1$
UP-B	

2.2.4 Correction factor F_{long}

The correction factor F_{long} depending on the edge stiffener types on the longer side of the buckling panel is defined in Tab 3. An average value of F_{long} is to be used for the plate panels having different edge stiffeners. For stiffener types other than those mentioned in Tab 3, the value of c is to be agreed by the Society. In such a case, a value of c higher than those mentioned in Tab 3 can be used, provided it is verified by buckling strength check of panel using non-linear FEA and deemed appropriate by the Society.

2.2.5 Correction factor F_{tran}

The correction factor F_{tran} is to be taken as:

- For transversely framed EPP of single-side skin ships, between the hopper and top wing tank:
  - when the two adjacent frames are supported by one tripping bracket fitted in way of the adjacent plate panels:

    $F_{tran} = 1{,}25$
  - when the two adjacent frames are supported by two tripping brackets each fitted in way of the adjacent plate panels:

    $F_{tran} = 1{,}33$
  - elsewhere:

    $F_{tran} = 1{,}15$
- For other cases:

  $F_{tran} = 1$

2.2.6 Curved plate panels

This requirement for curved plate limit state is applicable when $R/t_p \le 2500$. Otherwise, the requirement for plate limit state given in [2.2.1] is applicable.

The curved plate limit state is based on the following interaction formula:

$$\left(\frac{\gamma_c \sigma_{ax} S}{C_{ax} R_{eH\_P}}\right)^{1,25} + \left(\frac{\gamma_c \sigma_{tg} S}{C_{tg} R_{eH\_P}}\right)^{1,25} + \left(\frac{\gamma_c \tau \sqrt{3} S}{C_\tau R_{eH\_P}}\right)^{2} - \Upsilon = 1$$

with:

$$\Upsilon = 0,5\left(\frac{\gamma_c \sigma_{ax} S}{C_{ax} R_{eH\_P}}\right)\left(\frac{\gamma_c \sigma_{tg} S}{C_{tg} R_{eH\_P}}\right)$$

where:

σ_{ax} : Axial stress applied to the cylinder corresponding to the curved plate panel, in N/mm².
In case of tensile axial stresses: $\sigma_{ax} = 0$

σ_{tg} : Tangential stress applied to the cylinder corresponding to the curved plate panel, in N/mm².
In case of tensile tangential stresses: $\sigma_{tg} = 0$

C_{ax}, C_{tg}, C_τ : Buckling reduction factors of the curved plate panel, as defined in Tab 5.

The stress multiplier factor γ_c of the curved plate panel need not be taken less than the stress multiplier factor γ_c obtained from [2.2.1] for an expanded plane panel.

2.2.7 Normal stresses applied to plate panel

The normal stresses σ_x and σ_y, in N/mm², to be applied for the plate panel capacity calculation as given in [2.2.1], are to be taken as follows:

- for FE analysis, the reference stresses as defined in Sec 4, [2.4]
- for prescriptive assessment, the axial or transverse compressive stresses at load calculation points of the considered elementary plate panel, as defined in the applicable Rules
- for grillage analysis where the stresses are obtained based on the beam theory, the following values:

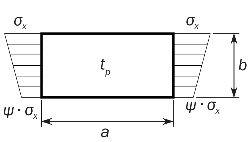
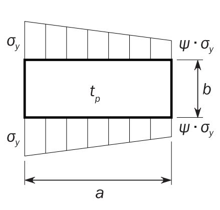
$$\sigma_x = \frac{\sigma_{xb} + \nu \sigma_{yb}}{1 - \nu^2}$$

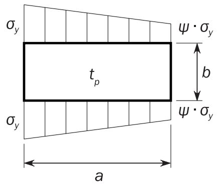
$$\sigma_y = \frac{\sigma_{yb} + \nu \sigma_{xb}}{1 - \nu^2}$$

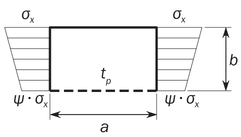
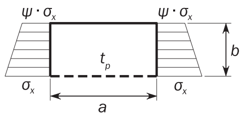
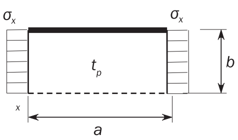
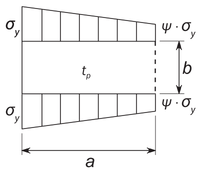
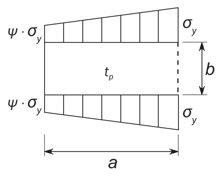
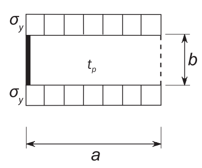
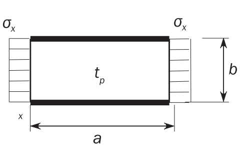
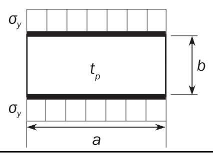
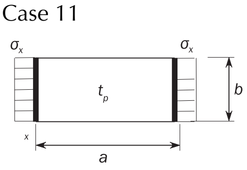
where:

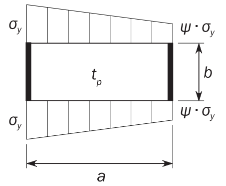
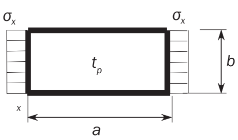
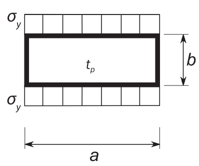
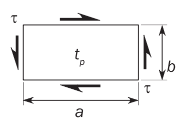
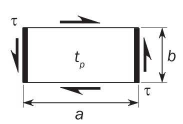
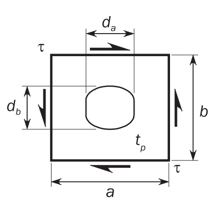
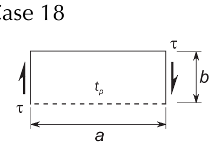
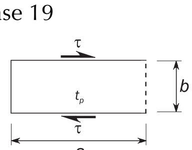
σ_{xb}, σ_{yb} : Stresses, in N/mm², from grillage beam analysis, respectively along x axis and y axis of the attached buckling panel.

Table 3 : Buckling factor K and reduction factor C for plane plate panels

Case	Stress ratio ψ	Buckling factor K	Reduction factor C
Case 1 	$1 \ge \psi \ge 0$	$K_x = F_{long}\frac{8,4}{\psi + 1,1}$	• when $\sigma_x \le 0$: $C_x = 1,00$ • when $\sigma_x > 0$: $C_x = 1,00$ for $\lambda \le \lambda_c$ $C_x = c\left(\frac{1}{\lambda} - \frac{0,22}{\lambda^2}\right)$ for $\lambda > \lambda_c$ where: $c = (1,25 - 0,12\psi) \le 1,25$ $\lambda_c = \frac{c}{2}\left(1 + \sqrt{1 - \frac{0,88}{c}}\right)$
	$0 > \psi > -1$	$K_x = F_{long}\,[7,63 - \psi\,(6,26 - 10\,\psi)]$	
	$\psi \le -1$	$K_x = F_{long}\,[5,975\,(1 - \psi)^2]$	
Case 2 	$1 \ge \psi \ge 0$	$K_y = F_{tran}\frac{2\left(1 + \frac{1}{\alpha^2}\right)^2}{1 + \psi + \frac{(1-\psi)}{100}\left(\frac{2,4}{\alpha^2} + 6,9f_1\right)}$ • when $\alpha \le 6$: $f_1 = (1-\psi)(\alpha - 1)$ • when $\alpha > 6$: $f_1 = 0,6\left(1 - \frac{6\psi}{\alpha}\right)\left(\alpha + \frac{14}{\alpha}\right)$ with $f_1 \le 14,5 - \frac{0,35}{\alpha^2}$	

Case	Stress ratio ψ	Buckling factor K	Reduction factor C
Case 2 (continued)  	$0 > \psi \geq 1 - \frac{4\alpha}{3}$	$K_y = \frac{200F_{tran}(1+\beta^2)^2}{(1-f_3)(100+2,4\beta^2+6,9f_1+23f_2)}$ • when $\alpha > 6\ (1 - \Psi)$: $f_1 = 0,6\left(\frac{1}{\beta}+14\beta\right)$ with $f_1 \leq 14{,}5 - 0{,}35\ \beta^2$ $f_2 = f_3 = 0$ • when $3\ (1 - \Psi) \leq \alpha \leq 6\ (1 - \Psi)$: $f_1 = \frac{1}{\beta} - 1$ $f_2 = f_3 = 0$ • when $1{,}5\ (1 - \Psi) \leq \alpha < 3\ (1 - \Psi)$: $f_1 = \frac{1}{\beta} - (2-\omega\beta)^4 - 9(\omega\beta-1)\left(\frac{2}{3}-\beta\right)$ $f_2 = f_3 = 0$ • when $1 - \Psi \leq \alpha < 1{,}5\ (1 - \Psi)$: - for $\alpha > 1{,}5$: $f_1 = 2\left[\frac{1}{\beta} - 16\left(1-\frac{\omega}{3}\right)^4\right]\left(\frac{1}{\beta}-1\right)$ $f_2 = 3\ \beta - 2$ $f_3 = 0$ - for $\alpha \leq 1{,}5$: $f_1 = 2\left(\frac{1,5}{1-\psi}-1\right)\left(\frac{1}{\beta}-1\right)$ $f_2 = \frac{\psi(1-16f_4^2)}{1-\alpha}$ $f_3 = 0$ $f_4 = [1{,}5 - \mathrm{Min}\ (1{,}5\ ;\ \alpha)]^2$ • when $0{,}75\ (1 - \Psi) \leq \alpha < 1 - \Psi$: $f_1 = 0$ $f_2 = 1 + 2,31(\beta-1) - 48\left(\frac{4}{3}-\beta\right)f_4^2$ $f_3 = 3f_4(\beta-1)\left(\frac{f_4}{1,81}-\frac{\alpha-1}{1,31}\right)$ $f_4 = [1{,}5 - \mathrm{Min}(1{,}5\ ;\ \alpha)]^2$	• when $\sigma_y \leq 0$: $C_y = 1{,}00$ • when $\sigma_y > 0$: $C_y = c\left[\frac{1}{\lambda} - \frac{R+F^2\ (H-R)}{\lambda^2}\right]$ where: $c = (1{,}25 - 0{,}12\ \Psi) \leq 1{,}25$ $R = \lambda\left(1-\frac{\lambda}{c}\right)$ for $\lambda < \lambda_c$ $R = 0{,}22$ for $\lambda \geq \lambda_c$ $\lambda_c = \frac{c}{2}\left(1+\sqrt{1-\frac{0,88}{c}}\right)$ $F = \left[1 - \frac{\left(\frac{K}{0,91}-1\right)}{\lambda_p^2}\right]c_1 \geq 0$ $\lambda_p^2 = \lambda^2 - 0,5$ for $1 \leq \lambda_p^2 \leq 3$ c_1 as defined in Tab 2 $H = \lambda - \frac{2\lambda}{c\ (T+\sqrt{T^2-4})} \geq R$ $T = \lambda + \frac{14}{15\lambda} + \frac{1}{3}$
	$\psi < 1 - \frac{4\alpha}{3}$	$K_y = 5,972F_{tran}\frac{\beta^2}{1-f_3}$ with: $f_3 = f_5\left(\frac{f_5}{1,81}+\frac{1+3\psi}{5,24}\right)$ $f_5 = \frac{9}{16}[1+\mathrm{Max}(-1;\psi)]^2$	

Case	Stress ratio ψ	Buckling factor K	Reduction factor C
Case 3 	$1 \geq \psi \geq 0$	$K_x = \dfrac{4\left(0,425 + \dfrac{1}{\alpha^2}\right)}{3\psi + 1}$	
	$0 > \psi \geq -1$	$K_x = 4\left(0,425 + \dfrac{1}{\alpha^2}\right)(1 + \psi) - 5\psi\,(1 - 3,42\psi)$	
Case 4 	$1 \geq \psi \geq -1$	$K_x = \left(0,425 + \dfrac{1}{\alpha^2}\right)\dfrac{3 - \psi}{2}$	$C_x = 1,00$ for $\lambda \leq 0,7$ $C_x = \dfrac{1}{\lambda^2 + 0,51}$ for $\lambda > 0,7$
Case 5 	–	• when $\alpha \geq 1,64$: $K_x = 1,28$ • when $\alpha < 1,64$: $K_x = \dfrac{1}{\alpha^2} + 0,56 + 0,13\alpha^2$	
Case 6 	$1 \geq \psi \geq 0$	$K_y = \dfrac{4(0,425 + \alpha^2)}{(3\psi + 1)\alpha^2}$	
	$0 > \psi \geq -1$	$K_y = 4(0,425 + \alpha^2)(1 + \psi)\dfrac{1}{\alpha^2} - 5\psi[1 - (3,42\psi)]\dfrac{1}{\alpha^2}$	
Case 7 	$1 \geq \psi \geq -1$	$K_y = (0,425 + \alpha^2)\dfrac{(3 - \psi)}{2\alpha^2}$	$C_y = 1,00$ for $\lambda \leq 0,7$ $C_y = \dfrac{1}{\lambda^2 + 0,51}$ for $\lambda > 0,7$
Case 8 	–	$K_y = 1 + \dfrac{0,56}{\alpha^2} + \dfrac{0,13}{\alpha^4}$	
Case 9 	–	$K_x = 6,97$	$C_x = 1,00$ for $\lambda \leq 0,83$ $C_x = 1,13\left(\dfrac{1}{\lambda} - \dfrac{0,22}{\lambda^2}\right)$ for $\lambda > 0,83$
Case 10 	–	$K_y = 4 + \dfrac{2,07}{\alpha^2} + \dfrac{0,67}{\alpha^4}$	$C_y = 1,00$ for $\lambda \leq 0,83$ $C_y = 1,13\left(\dfrac{1}{\lambda} - \dfrac{0,22}{\lambda^2}\right)$ for $\lambda > 0,83$
Case 11 	–	• when $\alpha \geq 4$: $K_x = 4$ • when $\alpha < 4$: $K_x = 4 + 2,74\left(\dfrac{4 - \alpha}{3}\right)^4$	$C_x = 1,00$ for $\lambda \leq 0,83$ $C_x = 1,13\left(\dfrac{1}{\lambda} - \dfrac{0,22}{\lambda^2}\right)$ for $\lambda > 0,83$

Case	Stress ratio ψ	Buckling factor K	Reduction factor C
Case 12 	–	$K_y = K_{y2}$ with: K_{y2} : K_y determined as per Case 2	$C_y = C_{y2}$ for $\alpha < 2$ $C_y = \left(1,06 + \dfrac{1}{10\alpha}\right) C_{y2}$ for $\alpha \geq 2$ where: $C_{y2} = C_y$ determined as per Case 2
Case 13 	–	• when $\alpha \geq 4$: $K_x = 6{,}97$ • when $\alpha < 4$: $K_x = 6,97 + 3,1\left(\dfrac{4-\alpha}{3}\right)^4$	$C_x = 1,00$ for $\lambda \leq 0,83$ $C_x = 1,13\left(\dfrac{1}{\lambda} - \dfrac{0,22}{\lambda^2}\right)$ for $\lambda > 0,83$
Case 14 	–	$K_y = \dfrac{6,97}{\alpha^2} + \dfrac{3,1}{\alpha^2}\left(\dfrac{4 - \dfrac{1}{\alpha}}{3}\right)^4$	$C_y = 1,00$ for $\lambda \leq 0,83$ $C_y = 1,13\left(\dfrac{1}{\lambda} - \dfrac{0,22}{\lambda^2}\right)$ for $\lambda > 0,83$
Case 15 	–	$K_\tau = \sqrt{3}\left(5,34 + \dfrac{4}{\alpha^2}\right)$	$C_\tau = 1,00$ for $\lambda \leq 0,84$ $C_\tau = \dfrac{0,84}{\lambda}$ for $\lambda > 0,84$
Case 16 	–	$K_\tau = \sqrt{3}\left[5,34 + \mathrm{Max}\left(\dfrac{4}{\alpha^2} ; \dfrac{7,15}{\alpha^{2,5}}\right)\right]$	
Case 17 	–	$K_\tau = \sqrt{3}\left(5,34 + \dfrac{4}{\alpha^2}\right) r$ with: r : Opening reduction factor taken as: $r = \left(1 - \dfrac{d_a}{a}\right)\left(1 - \dfrac{d_b}{b}\right)$ with $\dfrac{d_a}{a} \leq 0,7$ and $\dfrac{d_b}{b} \leq 0,7$	
Case 18 		$K_\tau = \sqrt{3}\left(0,6 + \dfrac{4}{\alpha^2}\right)$	
Case 19 		$K_\tau = 8$	

Edge boundary conditions:

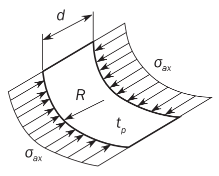
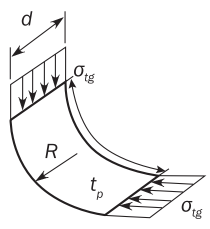
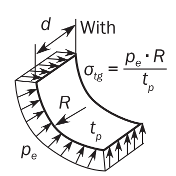
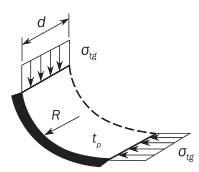
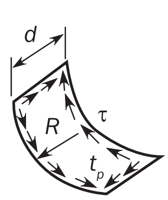
- - - - - - - - - Plate edge free.

——————— Plate edge simply supported.

▬▬▬▬▬▬ Plate edge clamped.

Note 1: The cases listed are general cases. Each stress component (σ_x, σ_y) is to be understood in local coordinates.

Table 4 : Buckling factor K and reduction factor C for curved plate panel with $R/t_p \le 2500$

Case	Aspect ratio	Buckling factor K	Reduction factor C
Case 1 	$\frac{d}{R} \le 0,5\sqrt{\frac{R}{t_p}}$	$K = 1 + \frac{2}{3}\,\frac{d^2}{Rt_p}$	• for general application: $C_{ax} = 1,00$ for $\lambda \le 0,25$ $C_{ax} = 1,233 - 0,933\,\lambda$ for $0,25 < \lambda \le 1,0$ $C_{ax} = \frac{0,30}{\lambda^3}$ for $1,0 < \lambda \le 1,5$ $C_{ax} = \frac{0,20}{\lambda^2}$ for $\lambda > 1,5$ • for curved single fields, e.g. bilge strakes, which are bounded by plane panels: $C_{ax} = \frac{0,65}{\lambda^2} \le 1,0$
	$\frac{d}{R} > 0,5\sqrt{\frac{R}{t_p}}$	$K = 0,267\,\frac{d^2}{Rt_p}\left[3 - \frac{d}{R}\sqrt{\frac{t_p}{R}}\right] \ge 0,4\,\frac{d^2}{Rt_p}$	
Case 2a 	$\frac{d}{R} \le 1,63\sqrt{\frac{R}{t_p}}$	$K = \frac{d}{\sqrt{Rt_p}} + 3\,\frac{(Rt_p)^{0,175}}{d^{0,35}}$	• for general application: $C_{tg} = 1,00$ for $\lambda \le 0,4$ $C_{tg} = 1,274 - 0,686\,\lambda$ for $0,4 < \lambda \le 1,2$ $C_{tg} = \frac{0,65}{\lambda^2}$ for $\lambda > 1,2$ • for curved single fields, e.g. bilge strakes, which are bounded by plane panels: $C_{tg} = \frac{0,8}{\lambda^2} \le 1,0$
Case 2b  p_e = external pressure in [N/mm²]	$\frac{d}{R} > 1,63\sqrt{\frac{R}{t_p}}$	$K = 0,3\,\frac{d^2}{R^2} + 2,25\left(\frac{R^2}{d\,t_p}\right)^2$	
Case 3 	$\frac{d}{R} \le \sqrt{\frac{R}{t_p}}$	$K = \frac{0,6d}{\sqrt{Rt_p}} + \frac{\sqrt{Rt_p}}{d} - 0,3\,\frac{Rt_p}{d^2}$	as in Case 2a
	$\frac{d}{R} > \sqrt{\frac{R}{t_p}}$	$K = 0,3\,\frac{d^2}{R^2} + 0,291\left(\frac{R^2}{d\,t_p}\right)^2$	
Case 4 	$\frac{d}{R} \le 8,7\sqrt{\frac{R}{t_p}}$	$K = \sqrt{3}\sqrt{28,3 + \frac{0,67\,d^3}{R^{1,5}\,t_p^{1,5}}}$	$C_\tau = 1,00$ for $\lambda \le 0,4$ $C_\tau = 1,274 - 0,686\,\lambda$ for $0,4 < \lambda \le 1,2$ $C_\tau = \frac{0,65}{\lambda^2}$ for $\lambda > 1,2$
	$\frac{d}{R} > 8,7\sqrt{\frac{R}{t_p}}$	$K = \sqrt{3}\,\frac{0,28d^2}{R\sqrt{Rt_p}}$	

Edge boundary conditions:

- - - - - - - - - Plate edge free.

———————— Plate edge simply supported.

▬▬▬▬▬▬ Plate edge clamped.

The shear stress τ, in N/mm², to be applied for the plate panel capacity calculation as given in [2.2.1], is to be taken as follows:

- for FE analysis, the reference shear stresses as defined in Sec 4, [2.4]
- for prescriptive assessment, the shear stresses at load calculation points of the considered elementary plate panel, as defined in the applicable Rules
- for grillage beam analysis, $\tau = 0$ in the attached buckling panel.

2.3 Stiffeners

2.3.1 Buckling modes

The following buckling modes are to be checked:

- stiffener induced failure (SI)
- associated plate induced failure (PI).

2.3.2 Effective web thickness of flat bars

For accounting the decrease of stiffness due to local lateral deformation in the case of flat bars, their net sectional area A_s, net section modulus Z and moment of inertia I, when applied in the formulae of [2.3.4], are to be calculated using, instead of t_w, the effective web thickness t_{w_red}, in mm, equal to:

$$t_{w\_red} = t_w\left[1 - \frac{2\pi^2}{3}\left(\frac{h_w}{s}\right)^2\left(1 - \frac{b_{eff1}}{s}\right)\right]$$

2.3.3 Idealisation of bulb bars

Bulb bars are to be considered as equivalent angle bars, as defined in the applicable Rules.

2.3.4 Ultimate buckling capacity

When $\sigma_a + \sigma_b + \sigma_w > 0$, the ultimate buckling capacity for stiffeners is to be checked according to the following interaction formula:

$$\frac{\gamma_c\ \sigma_a + \sigma_b + \sigma_w}{R_{eH}}\ S = 1$$

where:

σ_a : Effective axial stress, in N/mm², at mid span of the stiffener, acting on the stiffener with its attached plating:

$$\sigma_a = \sigma_x\ \frac{s\ t_p + A_s}{b_{eff1}\ t_p + A_s}$$

σ_x : Nominal axial stress, in N/mm², acting on the stiffener with its attached plating:

- for FE analysis, σ_x is the FE corrected stress, as defined in [2.3.6], in the attached plating in the direction of the stiffener axis
- for prescriptive assessment, σ_x is the axial stress at load calculation point of the stiffener, as defined in the applicable Rules
- for grillage beam analysis, σ_x is the stress acting along the x axis of the attached buckling panel

R_{eH} : Specified minimum yield stress of the material, in N/mm²:

- for stiffener induced failure (SI): $R_{eH} = R_{eH\_S}$
- for associated plate induced failure (PI): $R_{eH} = R_{eH\_P}$

σ_b : Bending stress in the stiffener, in N/mm²:

$$\sigma_b = \frac{M_0 + M_1}{1000Z}$$

Z : Net section modulus of the stiffener, in cm³, including effective width of the attached plating according to [2.3.5], to be taken as:

- the section modulus calculated at the top of the stiffener flange for stiffener induced failure (SI)
- the section modulus calculated at the attached plating for associated plate induced failure (PI)

C_{PI} : Associated plate induced failure pressure coefficient:

- $C_{PI} = 1$ if the lateral pressure is applied on the side opposite to the stiffener
- $C_{PI} = -1$ if the lateral pressure is applied on the same side as the stiffener

C_{SI} : Stiffener induced failure pressure coefficient:

- $C_{SI} = -1$ if the lateral pressure is applied on the side opposite to the stiffener
- $C_{SI} = 1$ if the lateral pressure is applied on the same side as the stiffener

M_1 : Bending moment, in N.mm, due to the lateral load P:

- for continuous stiffener:

$$M_1 = C_i\frac{|P|s\ell^2}{24 \cdot 10^3}$$

- for sniped stiffener:

$$M_1 = C_i\frac{|P|s\ell^2}{8 \cdot 10^3}$$

- for sniped stiffener at one end and continuous at the other end:

$$M_1 = C_i\frac{|P|s\ell^2}{14,2 \cdot 10^3}$$

P : Lateral load, in kN/m²:

- for FE analysis, P is the average pressure P_{avr} as defined in Sec 4, [2.5.2] in the attached plating
- for prescriptive assessment, P is the pressure calculated at load calculation point of the stiffener, as defined in the applicable Rules

C_i : Pressure coefficient:

- for stiffener induced failure (SI): $C_i = C_{SI}$
- for associated plate induced failure (PI): $C_i = C_{PI}$

M_0 : Bending moment, in N·mm, due to the lateral deformation w of the stiffener:

$$M_0 = F_E\left(\frac{P_z\ w}{c_f - P_z}\right)$$

with

$c_f - P_z > 0$

F_E : Ideal elastic buckling force of the stiffener, in N:

$$F_E = \left(\frac{\pi}{\ell}\right)^2 E\ I\ 10^4$$

I : Moment of inertia of the stiffener, in cm^4, including effective width of the attached plating according to [2.3.5].

I is to satisfy the following criterion:

$$I \geq \frac{s\ t_p^3}{12 \cdot 10^4}$$

t_p : Net thickness of the attached plating, in mm, to be taken as:

- for prescriptive requirements: the mean thickness of the two attached plating panels
- for FE analysis: the thickness of the considered EPP on one side of the stiffener

P_z : Nominal lateral load, in N/mm^2, acting on the stiffener due to stresses, σ_x , σ_y and τ, in the attached plating in way of the stiffener mid-span:

$$P_z = \frac{t_p}{s}\left[\sigma_{xl}\left(\frac{\pi\ s}{\ell}\right)^2 + 2c\ \gamma\ \sigma_y + \sqrt{2}\tau_1\right]$$

$$\sigma_{xl} = \gamma\ \sigma_x\left(1 + \frac{A_s}{s t_p}\right) \geq 0$$

$$\tau_1 = \gamma|\tau| - t_p\sqrt{R_{eH\_p}\ E\left(\frac{m_1}{a^2} + \frac{m_2}{b^2}\right)} \geq 0$$

σ_y : Stress applied on the edge along the y axis of the buckling panel, in N/mm^2, without being taken less than 0:

- for FE analysis, σ_y is the FE corrected stress, as defined in [2.3.6], in the attached plating in the direction perpendicular to the stiffener axis
- for prescriptive assessment, σ_y is the maximum compressive stress at load calculation points of the stiffener attached plating, as defined in the applicable Rules
- for grillage beam analysis, σ_y is the stress acting along the y axis of the attached buckling panel

τ : Applied shear stress, in N/mm^2:

- for FE analysis, τ is the reference shear stress, as defined in Sec 4, [2.4.2], in the attached plating
- for prescriptive assessment, τ is the shear stress of the stiffener attached plating, calculated according to the applicable Rules at the following load calculation points:
  - at the middle of the full span, ℓ, of the considered stiffener
  - at the intersection point between the stiffener and its attached plating.
- for grillage beam analysis, τ = 0 in the attached buckling panel

m_1 , m_2 : Coefficients taken equal to:

- when $\alpha \geq 2$: m_1 = 1,47 and m_2 = 0,49
- when $\alpha < 2$: m_1 = 1,96 and m_2 = 0,37

c : Factor taking into account the stresses in the attached plating acting perpendicular to the stiffener axis:

- for $0 \leq \psi \leq 1$: $c = 0,5\ (1 + \psi)$
- for $\psi < 0$: $c = \dfrac{1}{2(1-\psi)}$

ψ : Edge stress ratio for case 2 according to Tab 3

w : Deformation of stiffener, in mm: $w = w_0 + w_1$

w_0 : Assumed imperfection, in mm, to be taken as:

- in general:

  $w_0 = \ell / 1000$
- for stiffeners sniped at one or both ends, considering stiffener induced failure (SI):

  $w_0 = -\ w_{na}$
- for stiffeners sniped at one or both ends, considering associated plate induced failure (PI):

  $w_0 = w_{na}$

w_{na}: Distance from the mid-point of attached plating to the neutral axis of the stiffener calculated with the effective width of the attached plating according to [2.3.5]

w_1 : Deformation of the stiffener, in mm, at mid-point of the stiffener span, due to lateral load P.

In case of uniformly distributed load, w_1 is to be taken as:

- in general:

$$w_1 = C_i\ \frac{|P| s \ell^4}{384 \cdot 10^7\ EI}$$

- for stiffeners sniped at both ends:

$$w_1 = C_i\ \frac{5|P| s \ell^4}{384 \cdot 10^7\ EI}$$

- for stiffeners sniped at one end and continuous at the other end:

$$w_1 = C_i\ \frac{2|P| s \ell^4}{384 \cdot 10^7\ EI}$$

c_f : Elastic support provided by the stiffener, in N/mm^2:

$$c_f = F_E\left(\frac{\pi}{\ell}\right)^2 (1 + c_p)$$

with:

$$c_p = \frac{1}{1 + \frac{0,91}{c_{xa}}\left(\frac{12I}{s t_p^3}10^4 - 1\right)}$$

c_{xa} : Coefficient to be taken as:

- for $\ell \geq 2s$: $c_{xa} = \left(\dfrac{\ell}{2s} + \dfrac{2s}{\ell}\right)^2$
- for $\ell < 2s$: $c_{xa} = \left[1 + \left(\dfrac{\ell}{2s}\right)^2\right]^2$

Table 5 : Moments of inertia I_P, I_T and I_ω

Flat bars **(1)**	Bulb, angle, L2, L3 and T-bars	
$I_P = \dfrac{h_w^3\ t_w}{3 \cdot 10^4}$	$I_P = \left(\dfrac{A_w\ (e_f - 0,5t_f)^2}{3} + A_f\ e_f^2\right)\ 10^{-4}$	
$I_T = \dfrac{h_w\ t_w^3}{3 \cdot 10^4}\left(1 - 0,63\ \dfrac{t_w}{h_w}\right)$	$I_T = \dfrac{(e_f - 0,5t_f)\ t_w^3}{3 \cdot 10^4}\left(1 - 0,63\ \dfrac{t_w}{e_f - 0,5t_f}\right) + \dfrac{b_f\ t_f^3}{3 \cdot 10^4}\left(1 - 0,63\ \dfrac{t_f}{b_f}\right)$	
$I_\omega = \dfrac{h_w^3\ t_w^3}{36 \cdot 10^6}$	• for bulb, angle, L2 and L3 bars:	$I_\omega = \dfrac{A_f\ e_f^2\ b_f^2}{12 \cdot 10^6}\left(\dfrac{A_f + 2,6A_w}{A_f + A_w}\right)$
	• for T-bars:	$I_\omega = \dfrac{b_f^3\ t_f\ e_f^2}{12 \cdot 10^6}$
(1) t_w is the net web thickness, in mm (see Fig 1). t_{w_red} as defined in [2.3.2] is not to be used in this Table.		

σ_w : Stress due to torsional deformation, in N/mm², to be taken as:

- for stiffener induced failure (SI):

$$\sigma_w = E\ y_w\left(\frac{t_f}{2} + h_w\right)\Phi_0\left(\frac{\pi}{\ell}\right)^2\left(\frac{1}{1 - \dfrac{0,4R_{eH\_S}}{\sigma_{ET}}} - 1\right)$$

- for associated plate induced failure (PI):

$\sigma_w = 0$

y_w : Distance, in mm, from the centroid of the stiffener cross-section to the free edge of the stiffener flange, to be taken as:

- for flat bars:

$$y_w = \frac{t_w}{2}$$

- for angle and bulb bars:

$$y_w = b_f - \frac{h_w\ t_w^2 + t_f\ b_f^2}{2A_s}$$

- for T-bars:

$$y_w = \frac{b_f}{2}$$

- for L2 bars:

$$y_w = b_{f\text{-}out} + 0,5t_w - \frac{h_w\ t_w^2 + t_f(b_f^2 - 2b_fd_f)}{2A_s}$$

- for L3 bars:

$$y_w = b_{f\text{-}out} + 0,5t_w - \frac{(h_w - t_f)t_w^2 + t_f(b_f + t_w)^2}{2A_s}$$

Φ_0 : Coefficient taken as:

$$\Phi_0 = \frac{\ell}{h_w}\ 10^{-3}$$

σ_{ET} : Reference stress for torsional buckling, in N/mm²:

$$\sigma_{ET} = \frac{E}{I_P}\left(\frac{\varepsilon\pi^2\ I_\omega\ 10^2}{\ell^2} + 0,385I_T\right)$$

I_P : Net polar moment of inertia of the stiffener, in cm⁴, about point C (see Fig 1), as defined in Tab 5

I_T : Net Saint Venant's moment of inertia of the stiffener, in cm⁴, as defined in Tab 5

I_ω : Net sectional moment of inertia of the stiffener, in cm⁶, about point C (see Fig 1), as defined in Tab 5

ε : Degree of fixation:

$$\varepsilon = 1 + \frac{\left(\dfrac{\ell}{\pi}\right)^2\ 10^{-3}}{\sqrt{I_\omega\left(\dfrac{0,75s}{t_p^3} + \dfrac{e_f - 0,5t_f}{t_w^3}\right)}}$$

A_w : Net area of the stiffener web, in mm²

A_f : Net area of the stiffener flange, in mm².

2.3.5 Effective width of attached plating

The effective width b_{eff}, in mm, of the stiffener attached plating is to be taken as:

- when $\sigma_x > 0$:
  - for FE analysis:

    $b_{eff} = Min\ (C_x\ b\ ;\ \chi_s\ s)$
  - for prescriptive assessment:

$$b_{eff} = Min\left(\frac{C_{x1}\ b_1 + C_{x2}\ b_2}{2}\ ;\ \chi_s s\right)$$

- when $\sigma_x \le 0$:

  $b_{eff} = \chi_s\ s$

where:

χ_s : Effective width coefficient to be taken as:

- for $\ell_{eff}/s \ge 1$:

$$\chi_s = Min\left[\frac{1,12}{1 + \dfrac{1,75}{\left(\dfrac{\ell_{eff}}{s}\right)^{1,6}}}\ ;\ 1,0\right]$$

- for $\ell_{eff}/s < 1$:

$$\chi_s = 0,407\frac{\ell_{eff}}{s}$$

ℓ_{eff} : Effective length of the stiffener, in mm, taken as:

- for a stiffener fixed at both ends:

$$\ell_{eff} = \frac{\ell}{\sqrt{3}}$$

- for a stiffener simply supported at one end and fixed at the other:

$$\ell_{eff} = 0,75\ \ell$$

- for a stiffener simply supported at both ends:

$$\ell_{eff} = \ell$$

2.3.6 FE corrected stresses for stiffener capacity

When the reference stresses σ_x and σ_y obtained by FE analysis according to Sec 4, [2.4] are both compressive, they are to be corrected according to the following formulae:

- if $\sigma_x < \nu\ \sigma_y$:

  $\sigma_{xcor} = 0$

  $\sigma_{ycor} = \sigma_y$

- if $\sigma_y < \nu\ \sigma_x$:

  $\sigma_{xcor} = \sigma_x$

  $\sigma_{ycor} = 0$

- in the other cases:

  $\sigma_{xcor} = \sigma_x - \nu\ \sigma_y$

  $\sigma_{ycor} = \sigma_y - \nu\ \sigma_x$

2.4 Primary supporting members

2.4.1 Web plate in way of openings

The web plate of primary supporting members with openings is to be assessed for buckling based on the combined axial compressive and shear stresses.

The web plate adjacent to the opening on both sides is to be considered as individual unstiffened plate panels as shown in Tab 7.

The interaction formulae of [2.2.1] are to be used with:

- $\sigma_x = \sigma_{av}$
- $\sigma_y = 0$
- $\tau = \tau_{av}$

where:

σ_{av} : Weighted average compressive stress, in N/mm^2, in the area of web plate being considered, i.e. P1, P2 or P3 as shown in Tab 7

τ_{av} : Weighted average shear stress, in N/mm^2:

- for opening modelled in primary supporting members:

  τ_{av} is the weighted average shear stress in the area of web plate being considered, i.e. P1, P2 or P3 as shown in Tab 7

- for opening not modelled in primary supporting members:

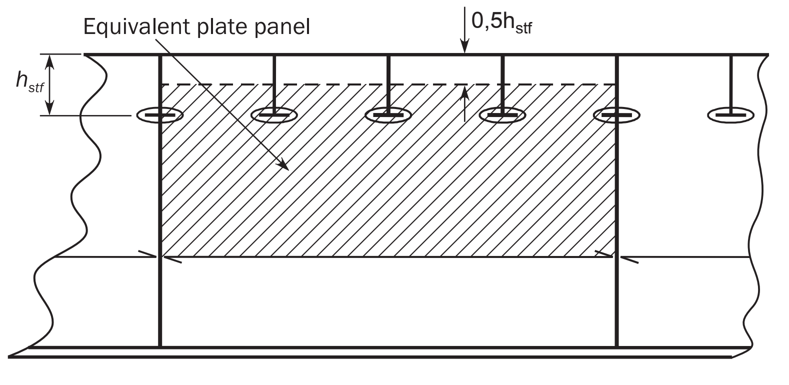
  τ_{av} is the weighted average shear stress given in Tab 7.

2.4.2 Reduction factors of web plate in way of openings

The reduction factors, C_x or C_y in combination with C_τ, of the plate panel(s) of the web adjacent to the opening is to be taken as shown in Tab 7.

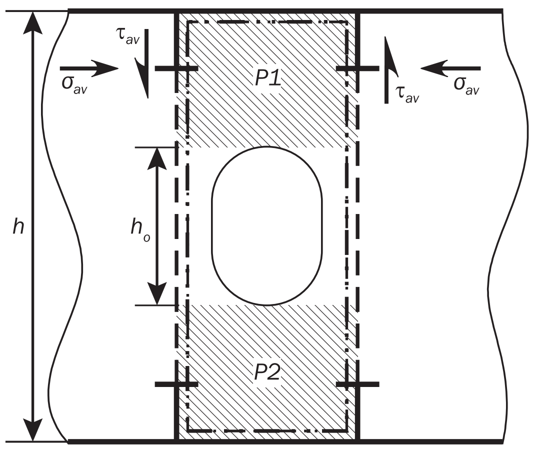
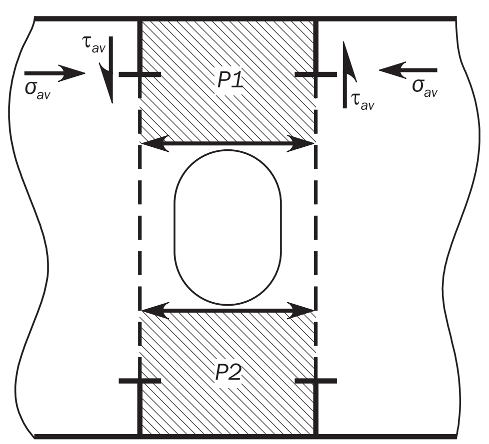
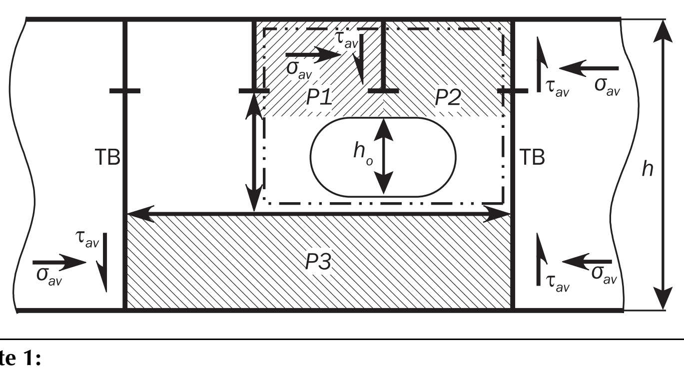
2.4.3 The equivalent plate panel of web plate of primary supporting members crossed by perpendicular stiffeners is to be idealised as shown in Fig 2.

Figure 2 : Web plate idealisation



The correction of panel breadth is also applicable for other slot configurations, provided the web or the collar plate is attached to at least one side of the passing stiffener.

Table 6 : Reduction factors C_x , C_y and C_τ

<table>
<tr><th rowspan="2">Configuration</th><th rowspan="2">C_x and C_y</th><th colspan="2">C_τ</th></tr>
<tr><th>Opening modelled in PSM</th><th>Opening not modelled in PSM</th></tr>
<tr><td>(a) Without edge reinforcements:
</td><td>Separate reduction factors are to be applied to areas P1 and P2 using case 3 or case 6 in Tab 3, with edge stress ratio $\psi = 1,0$</td><td>Separate reduction factors are to be applied to areas P1 and P2 using case 18 or case 19 in Tab 3</td><td>• when case 17 of Tab 3 is applicable:
A common reduction factor is to be applied to areas P1 and P2 using case 17 in Tab 3 with:
$\tau_{av} = \tau_{av}(web)$
• when case 17 of Tab 3 is not applicable:
Separate reduction factors are to be applied to areas P1 and P2 using case 18 or case 19 in Tab 3 with:
$\tau_{av} = \dfrac{\tau_{av}(web) \cdot h}{h - h_0}$</td></tr>
<tr><td>(b) With edge reinforcements:
</td><td>Separate reduction factors are to be applied to areas P1 and P2 using, in Tab 3 C_x for case 1 or C_y for case 2, with edge stress ratio $\psi = 1,0$</td><td>Separate reduction factors are to be applied to areas P1 and P2 using case 15 in Tab 3</td><td>Separate reduction factors are to be applied to areas P1 and P2 using case 15 in Tab 3, with:
$\tau_{av} = \dfrac{\tau_{av}(web) \cdot h}{h - h_0}$</td></tr>
<tr><td colspan="2">(c) Example of hole in web:
</td><td colspan="2">Panels P1 and P2 are to be evaluated in accordance with configuration (a)
Panel P3 is to be evaluated in accordance with configuration (b)</td></tr>
<tr><td colspan="4">Note 1:
h : Height, in m, of the web of the primary supporting member in way of the opening
h_0 : Height, in m, of the opening measured in the depth of the web
$\tau_{av}(web)$: Weighted average shear stress, in N/mm² over the web height h of the primary supporting member.
Note 2: Web panels to be considered for buckling in way of openings are shown shaded and numbered P1, P2, etc.</td></tr>
</table>

3 Buckling capacity of the other structures

3.1 Struts, pillars and cross ties

3.1.1 Buckling utilisation factor

The buckling utilisation factor η, for axially compressed struts, pillars and cross ties, is to be taken as:

$$\eta = \frac{\sigma_{av}}{\sigma_{cr}}$$

where:

σ_{av} : Average axial compressive stress in the member, in N/mm²

σ_{cr} : Minimum critical buckling stress, in N/mm², taken as:

- for $\sigma_E \leq 0{,}5\ R_{eH\_S}$:

  $\sigma_{cr} = \sigma_E$

- for $\sigma_E > 0{,}5\ R_{eH\_S}$:

  $$\sigma_{cr} = \left(1 - \frac{R_{eH\_S}}{4\sigma_E}\right) R_{eH\_S}$$

σ_E : Minimum elastic buckling stress, in N/mm², according to [3.1.2] to [3.1.6], as applicable.

R_{eH_S} : Specified minimum yield stress of the considered member, in N/mm². For built-up members, the lowest specified minimum yield stress is to be used.

3.1.2 Elastic column buckling stress

The elastic compressive column buckling stress σ_{EC} , in N/mm², of members subject to axial compression is to be taken as:

$$\sigma_{EC} = \pi^2\ Ef_{end}\ \frac{I}{A\ \ell_{pill}^2}\ 10^{-4}$$

where:

A : Net cross-sectional area of the member, in cm²

I : Net moment of inertia about the weakest axis of the cross-section, in cm⁴

ℓ_{pill} : Length of the member, in m:

- for pillars and struts:

  ℓ_{pill} is the unsupported length of the member

- for cross ties:
  - in centre tanks: ℓ_{pill} is the distance between the flanges of longitudinal stiffeners on the starboard and port longitudinal bulkheads to which the cross tie's horizontal stringer is attached
  - in wing tanks: ℓ_{pill} is the distance between the flanges of longitudinal stiffeners on the longitudinal bulkhead to which the cross tie's horizontal stringer is attached, and the inner hull plating.

f_{end} : End constraint factor, taken as:

- for pillars and struts:
  - f_{end} = 1,0 where both ends are simply supported
  - f_{end} = 2,0 where one end is simply supported and the other end is fixed
  - f_{end} = 4,0 where both ends are fixed
- for cross ties:

  f_{end} = 2,0

A pillar end may be considered fixed when brackets of adequate size are fitted. Such brackets are to be supported by structural members with bending stiffness greater than the pillar.

3.1.3 Elastic torsional buckling stress of open-type cross-sections

The elastic torsional buckling stress σ_{ET} , in N/mm², with respect to axial compression of members is to be taken as:

$$\sigma_{ET} = \frac{G I_{sv}}{I_{pol}} + \frac{\pi^2\ f_{end}\ E c_{warp}}{I_{pol}\ \ell_{pill}^2}\ 10^{-4}$$

where:

I_{sv} : Net Saint Venant's moment of inertia, in cm⁴ (see Tab 8 for examples of cross-sections)

I_{pol} : Net polar moment of inertia about the shear centre of cross-section, in cm⁴, taken as:

$$I_{pol} = I_y + I_z + A\ (y_0^2 + z_0^2)$$

c_{warp} : Warping constant, in cm⁶ (see Tab 8 for examples of cross-sections)

ℓ_{pill} : Length of the member, in m, as defined in [3.1.2]

I_y : Net moment of inertia about the y axis, in cm⁴

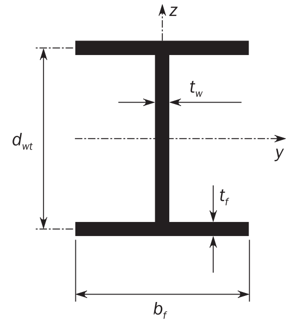
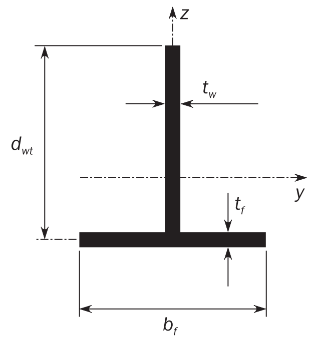
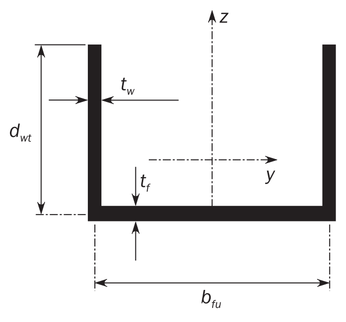
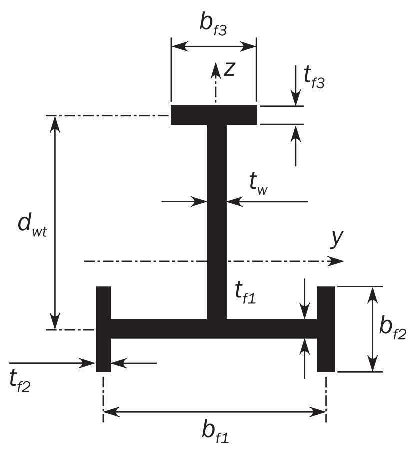
I_z : Net moment of inertia about the z axis, in cm⁴

A : Net cross-sectional area of the member, in cm²

y_0 : Transverse position of shear centre relative to the cross-sectional centroid, in cm (see Tab 8 for examples of cross-sections)

z_0 : Vertical position of shear centre relative to the cross-sectional centroid, in cm (see Tab 8 for examples of cross-sections).

Table 7 : Cross-sectional properties

Typical cross-sections	Properties	Units
	$I_{sv} = \frac{1}{3}(2 b_f t_f^3 + d_{wt} t_w^3) 10^{-4}$	cm^4
	$c_{warp} = \frac{d_{wt}^2 b_f^3 t_f}{24} 10^{-6}$	cm^6
	$I_{sv} = \frac{1}{3}(b_f t_f^3 + d_{wt} t_w^3) 10^{-4}$	cm^4
	$y_0 = 0$	cm
	$z_0 = -\frac{0,5 d_{wt}^2 t_w}{d_{wt} t_w + b_f t_f} 10^{-1}$	cm
	$c_{warp} = \frac{b_f^3 t_f^3 + 4 d_{wt}^3 t_w^3}{144} 10^{-6}$	cm^6
	$I_{sv-n50} = \frac{1}{3}(b_{fu} t_f^3 + 2 d_{wt} t_w^3) 10^{-4}$	cm^4
	$y_0 = 0$	cm
	$z_0 = -\frac{d_{wt}^2 t_w 10^{-1}}{2 d_{wt} t_w + b_{fu} t_f} - \frac{0,5 d_{wt}^2 t_w 10^{-1}}{d_{wt} t_w + b_{fu} t_f/6}$	cm
	$c_{warp} = \frac{b_{fu}^2 d_{wt}^3 t_w (3 d_{wt} t_w + 2 b_{fu} t_f)}{12(6 d_{wt} t_w + b_{fu} t_f)} 10^{-6}$	cm^6
	$I_{sv} = \frac{1}{3}(b_{f1} t_{f1}^3 + 2 b_{f2} t_{f2}^3 + b_{f3} t_{f3}^3 + d_{wt} t_w^3) 10^{-4}$	cm^4
	$y_0 = 0$	cm
	$z_0 = z_s - \frac{(b_{f3} d_{wt} t_{f3} + 0,5 d_{wt}^2 t_w) 10^{-1}}{d_{wt} t_w + b_{f1} t_{f1} + 2 b_{f2} t_{f2} + b_{f3} t_{f3}}$	cm
	$c_{warp} = \left[I_{f1} z_s^2 + \frac{I_{f2} b_{f1}^2}{200} + I_{f3}\left(\frac{d_{wt}}{10} - z_s\right)^2\right]$	cm^6
	$I_{f1} = \left[\frac{(b_{f1} - t_{f2})^3 t_{f1}}{12} + \frac{b_{f2} t_{f2} b_{f1}^2}{2}\right] 10^{-4}$	cm^4
	$I_{f2} = \frac{b_{f2}^3 t_{f2}}{12} 10^{-4}$	cm^4
	$I_{f3} = \frac{b_{f3}^3 t_{f3}}{12} 10^{-4}$	cm^4
	$z_s = \frac{I_{f3} d_{wt}}{I_{f1} + I_{f3}} 10^{-1}$	cm

Note 1: All the dimensions are in mm.
Note 2: Cross-sectional properties for cross-sections other than these typical ones are to be determined by direct calculation.

3.1.4 Elastic torsional/column buckling stress of open-type cross-sections

For the cross-sections where the centroid and the shear centre do not coincide, the interaction between the torsional and column buckling modes is to be examined.

The elastic torsional/column buckling stress σ_{ETF}, in N/mm², with respect to axial compression is to be taken as:

$$\sigma_{ETF} = \frac{1}{2\zeta}\left[(\sigma_{EC} + \sigma_{ET}) - \sqrt{(\sigma_{EC} + \sigma_{ET})^2 - 4\zeta\ \sigma_{EC}\ \sigma_{ET}}\right]$$

where:

σ_{EC} : Elastic compressive column buckling stress, as defined in [3.1.2]

σ_{ET} : Elastic torsional buckling stress, as defined in [3.1.3]

ζ : Coefficient taken as:

$$\zeta = 1 - \frac{(y_0^2 + z_0^2)\ A}{I_{pol}}$$

y_0, z_0, I_{pol} : As defined in [3.1.3]

A : Net cross-sectional area of the member, in cm².

3.1.5 Elastic local buckling stress of open-type cross-sections

The elastic local buckling stress σ_{EL1}, in N/mm², with respect to axial compression of open-type cross-sections is to be taken equal to the lesser of the values obtained from the following formulae:

- $\sigma_{EL1} = 78\left(\frac{t_W}{d_W}\right)^2 10^4$
- $\sigma_{EL1} = 32\left(\frac{t_f}{b_f}\right)^2 10^4$

3.1.6 Elastic local buckling stress of hollow rectangular cross-sections

The elastic local buckling stress σ_{EL2}, in N/mm², with respect to axial compression of hollow rectangular cross-sections is to be taken equal to the lesser of the values obtained from the following formulae:

- $\sigma_{EL2} = 78\left(\frac{t_2}{b}\right)^2 10^4$
- $\sigma_{EL2} = 78\left(\frac{t_1}{h}\right)^2 10^4$

b : Length, in mm, of the shorter side of the cross-section

t_2 : Net web thickness, in mm, of the shorter side of the cross-section

h : Length, in mm, of the longer side of the cross-section

t_1 : Net web thickness, in mm, of the longer side of the cross-section.

3.2 Corrugated bulkheads

3.2.1 The buckling utilisation factor of flange and web of corrugations of corrugated bulkheads is based on the combination of in-plane stresses and shear stress.

The interaction formulae of [2.2.1] are to be used considering the following coefficients:

- $\alpha = 2$
- $\psi_x = \psi_y = 1$

APPENDIX 1 STRESS BASED REFERENCE STRESSES

Symbols

a : Length, in mm, of the longer side of the plate panel as defined in Sec 5

b : Length, in mm, of the shorter side of the plate panel as defined in Sec 5

A_i : Area, in mm^2, of the i-th plate element of the buckling panel

n : Number of plate elements in the buckling panel

σ_{xi} : Actual stress, in N/mm^2, at the centroid of the i-th plate element in x direction, applied along the shorter edge of the buckling panel

σ_{yi} : Actual stress, in N/mm^2, at the centroid of the i-th plate element in y direction, applied along the longer edge of the buckling panel

ψ : Edge stress ratio as defined in Sec 5

τ_i : Actual membrane shear stress, in N/mm^2, at the centroid of the i-th plate element of the buckling panel.

1 Stress based method

1.1 Introduction

1.1.1 This Appendix provides a method to determine stress distribution along the edges of the considered buckling panel by 2nd order polynomial curve, by linear distribution using the least square method and by weighted average approach. This method is called Stress based method.

The reference stress is the stress components at centre of the plate element transferred into the local system of the considered buckling panel.

1.1.2 Definition

A regular panel is a plate panel of rectangular shape. An irregular panel is a plate panel which is not regular, as detailed in Sec 4, [2.3.1].

1.2 Stress application

1.2.1 Regular panel

The reference stresses are to be taken as defined in [2.1] for a regular panel when the following conditions are satisfied:

- at least one plate element centre is located in each third part of the long edge a of a regular panel, and
- this element centre is located at a distance in the panel local x direction not less than a/4 to at least one of the element centres in the adjacent third part of the panel.

Otherwise, the reference stresses are to be taken as defined in [2.2] for an irregular panel.

1.2.2 Irregular panel and curved panel

The reference stresses of an irregular panel or a curved panel are to be taken as defined in [2.2].

2 Reference stresses

2.1 Regular panel

2.1.1 Longitudinal stress

The longitudinal stress σ_x applied on the shorter edge of the buckling panel is to be calculated as follows:

- for plate buckling assessment, the distribution of $\sigma_x(x)$ is assumed as 2nd order polynomial curve:

  $\sigma_x(x) = C x^2 + D x + E$

  The best fitting curve $\sigma_x(x)$ is to be obtained by minimising the square error Π, considering the area of each element as a weighting factor:

$$\Pi = \sum_{i=1}^{n} A_i[\sigma_{xi} - (Cx_i^2 + Dx_i + E)]^2$$

  The unknown coefficients C, D and E must yield zero first partial derivatives $\partial\Pi$ with respect to C, D and E respectively:

$$\begin{cases} \dfrac{\partial\Pi}{\partial C} = 2\sum_{i=1}^{n} A_i x_i^2[\sigma_{xi} - (Cx_i^2 + Dx_i + E)] = 0 \\ \dfrac{\partial\Pi}{\partial D} = 2\sum_{i=1}^{n} A_i x_i[\sigma_{xi} - (Cx_i^2 + Dx_i + E)] = 0 \\ \dfrac{\partial\Pi}{\partial E} = 2\sum_{i=1}^{n} A_i[\sigma_{xi} - (Cx_i^2 + Dx_i + E)] = 0 \end{cases}$$

  The unknown coefficients C, D and E are obtained by solving the three above equations.

$$\sigma_{x1} = \frac{1}{b}\int_0^b \sigma_x(x)dx = \frac{b^2}{3}C + \frac{b}{2}D + E$$

$$\sigma_{x2} = \frac{1}{b}\int_{a-b}^{a} \sigma_x(x)dx = \left(a^2 - ab + \frac{b^2}{3}\right)C + \left(a - \frac{b}{2}\right)D + E$$

When (–D/2C < b/2) or (–D/2C > a – b/2), σ_{x3} is to be ignored. Otherwise:

$$\sigma_{x3} = \frac{1}{b}\int_{xmin}^{xmax} \sigma_x(x)dx = \frac{b^2}{12}C - \frac{D^2}{4C} + E$$

where:

$$x_{min} = -\frac{b}{2} - \frac{D}{2C}$$

$$x_{max} = \frac{b}{2} - \frac{D}{2C}$$

The longitudinal stress is to be taken as:

$\sigma_x = \text{Max}\ (\sigma_{x1}\ ;\ \sigma_{x2}\ ;\ \sigma_{x3})$

The edge stress ratio is to be taken as:

$\psi_x = 1$

- for stiffener buckling assessment, $\sigma_x(x)$ applied on the shorter edge of the attached plate is to be taken as:

$$\sigma_x = \frac{\sum_1^n A_i\ \sigma_{xi}}{\sum_1^n A_i}$$

The edge stress ratio ψ_x for the stress σ_x is equal to 1,0.

2.1.2 Transverse stress

The transverse stress σ_y applied along the longer edges of the buckling panel is to be calculated by extrapolation of the transverse stresses of all the elements up to the shorter edges of the considered buckling panel.

The distribution of $\sigma_y(x)$ is assumed to be a straight line.

Therefore:

$\sigma_y(x) = A + B\ x$

The best fitting curve $\sigma_y(x)$ is to be obtained by the least square method minimising the square error Π, considering the area of each element as a weighting factor:

$$\Pi = \sum_{i=1}^n A_i\ [\sigma_{yi} - (A + Bx_i)]^2$$

The unknown coefficients C and D must yield zero first partial derivatives $\partial\Pi$ with respect to C and D respectively:

$$\begin{cases} \dfrac{\partial\Pi}{\partial A} = 2\sum_{i=1}^n A_i\ [\sigma_{yi} - (A + Bx_i)] = 0 \\ \dfrac{\partial\Pi}{\partial B} = 2\sum_{i=1}^n A_i\ x_i\ [\sigma_{yi} - (A + Bx_i)] = 0 \end{cases}$$

The unknown coefficients A and B are obtained by solving the two previous equations, as follows:

$$\begin{cases} A = \dfrac{\left(\sum_{i=1}^n A_i\ \sigma_{yi}\right)\left(\sum_{i=1}^n A_i\ x_i^2\right) - \left(\sum_{i=1}^n A_i\ x_i\right)\left(\sum_{i=1}^n A_i\ x_i\ \sigma_{yi}\right)}{\left(\sum_{i=1}^n A_i\right)\left(\sum_{i=1}^n A_i\ x_i^2\right) - \left(\sum_{i=1}^n A_i\ x_i\right)^2} \\ B = \dfrac{\left(\sum_{i=1}^n A_i\right)\left(\sum_{i=1}^n A_i\ x_i\ \sigma_{yi}\right) - \left(\sum_{i=1}^n A_i\ x_i\right)\left(\sum_{i=1}^n A_i\ \sigma_{yi}\right)}{\left(\sum_{i=1}^n A_i\right)\left(\sum_{i=1}^n A_i\ x_i^2\right) - \left(\sum_{i=1}^n A_i\ x_i\right)^2} \end{cases}$$

$\sigma_y = \text{Max}\ (A\ ;\ A + Ba)$

$$\psi_y = \frac{\text{Min}\ (A;\ A + Ba)}{\text{Max}\ (A;\ A + Ba)} \quad \text{for } \sigma_y \geq 0$$

$\psi_y = 1{,}0$ for $\sigma_y < 0$

2.1.3 Shear stress

The shear stress τ is to be calculated using a weighted average approach and is to be taken as:

$$\tau = \frac{\sum_{i=1}^n A_i\ \tau_i}{\sum_{i=1}^n A_i}$$

2.2 Irregular panel and curved panel

2.2.1 Reference stresses

The longitudinal, transverse and shear stresses are to be calculated using a weighted average approach. They are to be taken as:

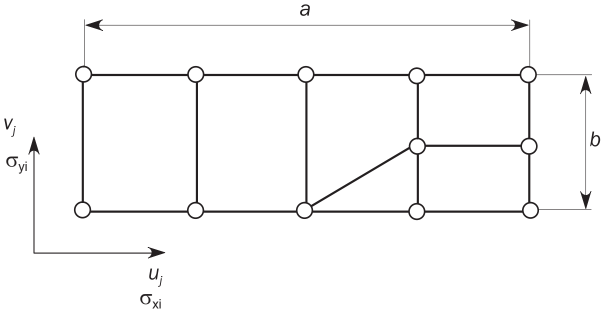
$$\sigma_x = \frac{\sum_{i=1}^n A_i\ \sigma_{xi}}{\sum_{i=1}^n A_i}\ ;\ \sigma_y = \frac{\sum_{i=1}^n A_i\ \sigma_{yi}}{\sum_{i=1}^n A_i}\ ;\ \tau = \frac{\sum_{i=1}^n A_i\ \tau_i}{\sum_{i=1}^n A_i}$$

The edge stress ratios are to be taken as follows:

$\psi_x = 1{,}0$

$\psi_y = 1{,}0$

Figure 1 : Buckling panel



APPENDIX 2 METHOD SELECTION FOR DIRECT STRENGTH ANALYSIS OF PANELS

1 Stiffened and unstiffened panels

1.1 General

1.1.1 This Appendix provides guidance for the selection of the modelling method of plate panels, when assessed for buckling through direct strength analysis according to Sec 4

1.1.2 The plate panels of hull structure are to be modelled as stiffened or unstiffened panels. Method A or Method B as defined in Sec 1, [2] is to be used according to Fig 1to Fig 11.

Table 1 : Structural members

Structural elements	Assessment method	Normal panel definition
Longitudinal structure (see Fig 1, Fig 2 and Fig 7)		
Longitudinally stiffened panels Shell envelope Deck Inner hull Hopper tank sides Longitudinal bulkheads	SP-A	length: between web frames width: between primary supporting members
Double bottom longitudinal girders in line with longitudinal bulkheads or connected to hopper tank sides	SP-A	length: between web frames width: full web depth
Web of double bottom longitudinal girders not in line with longitudinal bulkheads or not connected to hopper tank sides	SP-B	length: between web frames width: full web depth
Web of horizontal girders in double side spaces connected to hopper tank sides	SP-A	length: between web frames width: full web depth
Web of horizontal girders in double side spaces not connected to hopper tank sides	SP-B	length: between web frames width: full web depth
Web of single skin longitudinal girders or stringers	UP-B	plate between local stiffeners/face plate/PSM
Transverse structure (see Fig 3 Fig 4, Fig 5, Fig 8 and Fig 11)		
Web of transverse deck frames, including brackets	UP-B	plate between local stiffeners/face plate/PSM
Vertical web in double side spaces	SP-B	length: full web depth width: between primary supporting members
Irregularly stiffened panels, e.g. web panels in way of hopper tanks and bilges	UP-B	plate between local stiffeners/face plate/PSM
Double bottom floors	SP-B	length: full web depth width: between primary supporting members
Vertical web frames, including brackets	UP-B	plate between vertical web stiffeners/face plate/PSM
Cross tie web plates	UP-B	plate between vertical web stiffeners/face plate/PSM
Transverse watertight bulkheads (see Fig 6, Fig 9 and Fig 10)		
Regularly stiffened bulkhead panels including the secondary buckling stiffeners perpendicular to the regular stiffeners (such as carlings)	SP-A	length: between primary supporting members width: between primary supporting members
Irregularly stiffened bulkhead panels, e.g. web panels in way of hopper tanks and bilges	UP-B	plate between local stiffeners/face plate
Web plate of bulkhead stringers, including brackets	UP-B	plate between web stiffeners/face plate
Transverse corrugated bulkheads and cross deck		
Cross deck	SP-A	plate between local stiffeners/PSM
Note 1: SP, UP : Stiffened panel and Unstiffened panel, respectively A, B : Method A and Method B, respectively PSM : Primary supporting member.		

Figure 1 : Longitudinal plates for double bottom offshore units

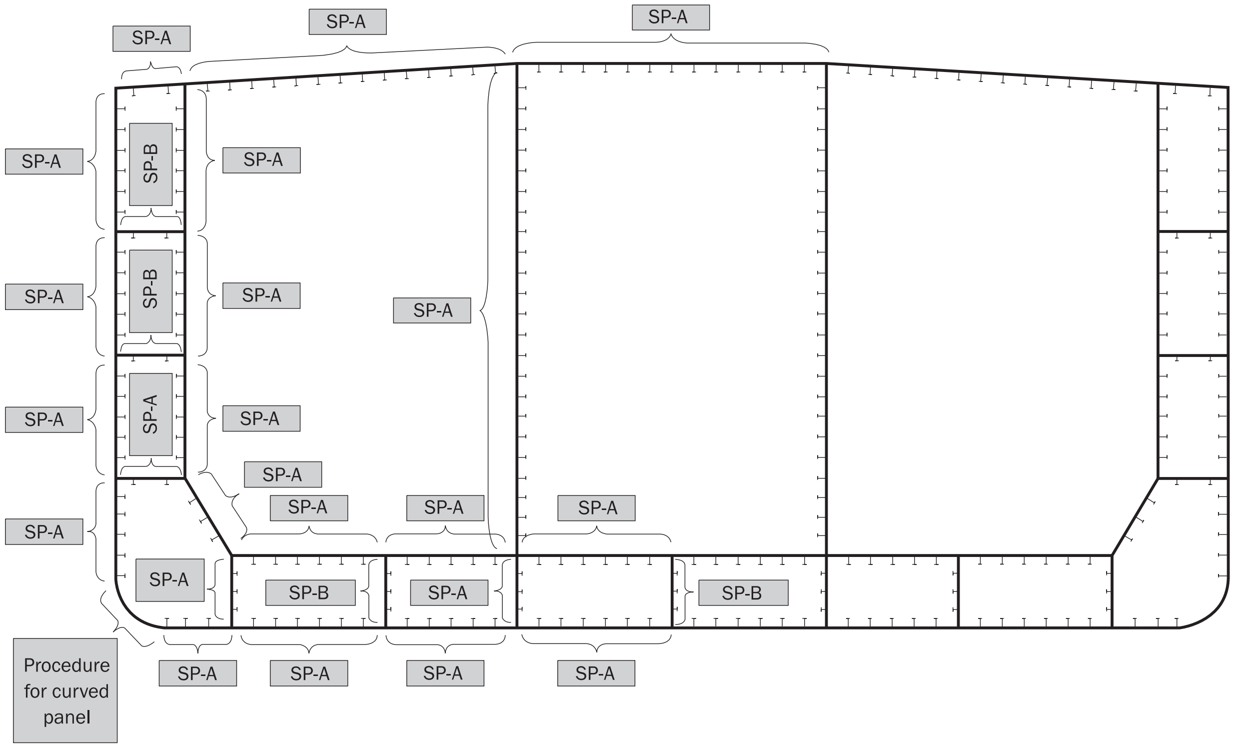


Figure 2 : Longitudinal plates for single bottom offshore units

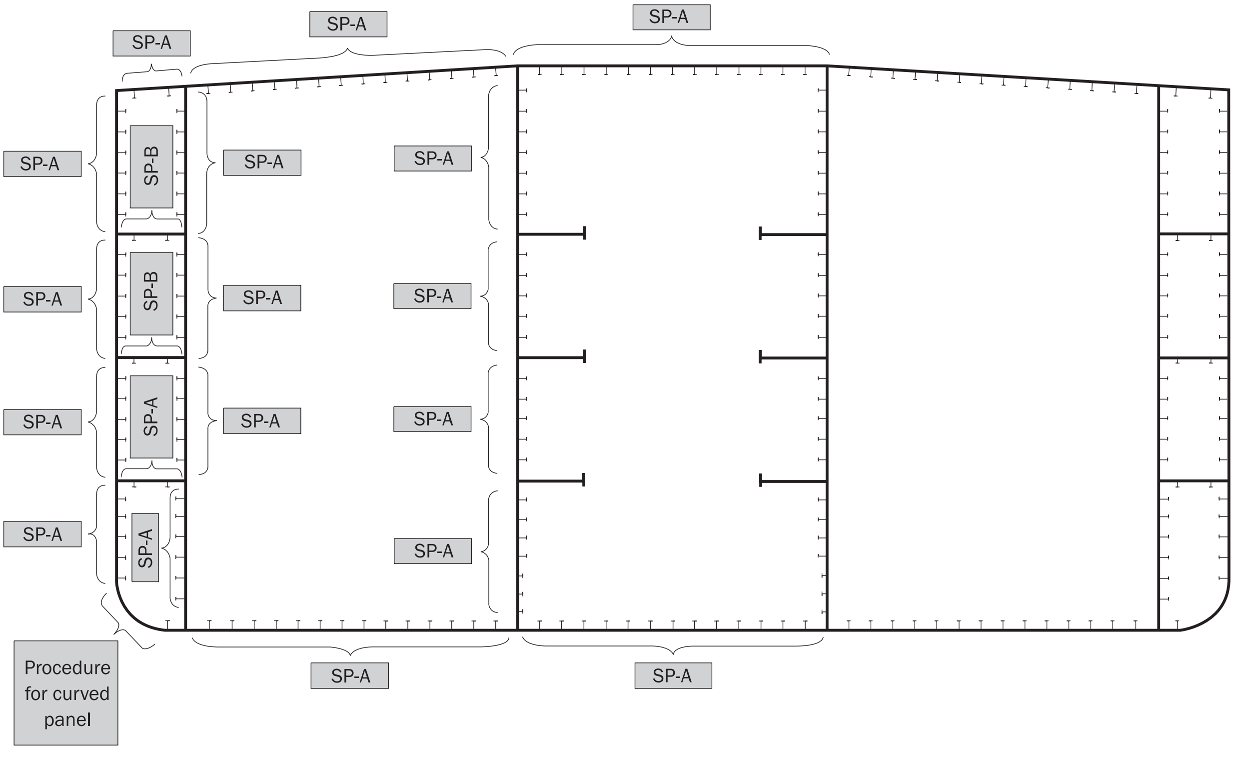


Figure 3 : Transverse web frames for double bottom offshore units

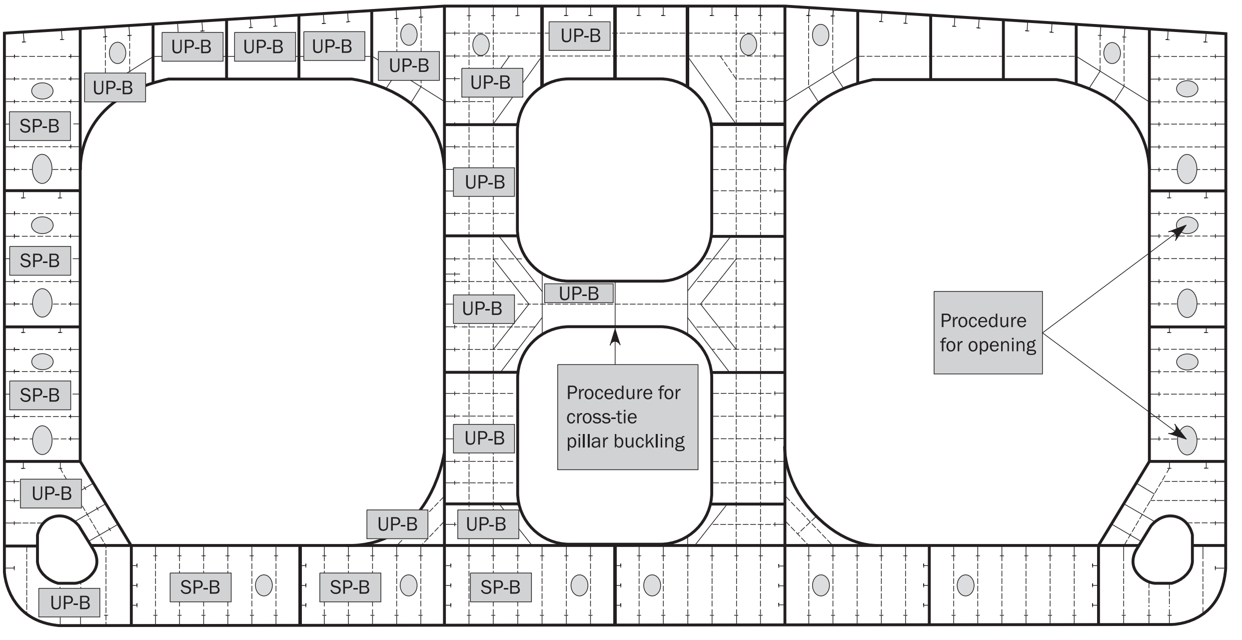


Figure 4 : Transverse web frames for single bottom offshore units

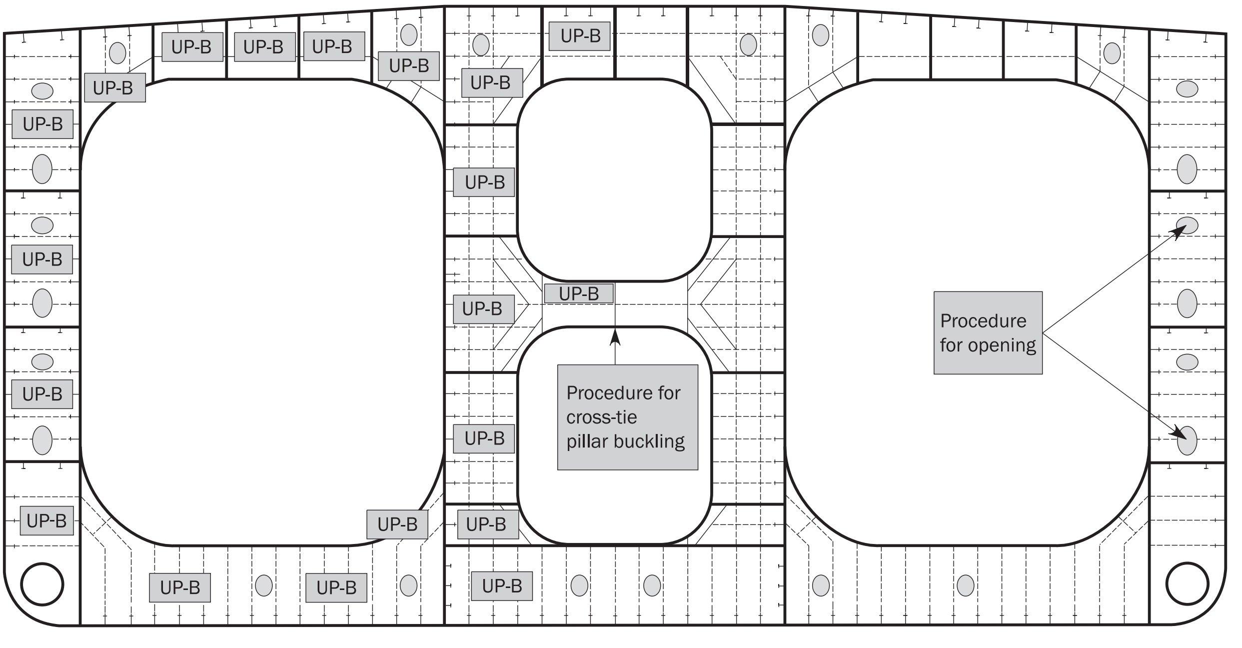


Figure 5 : Cross tie

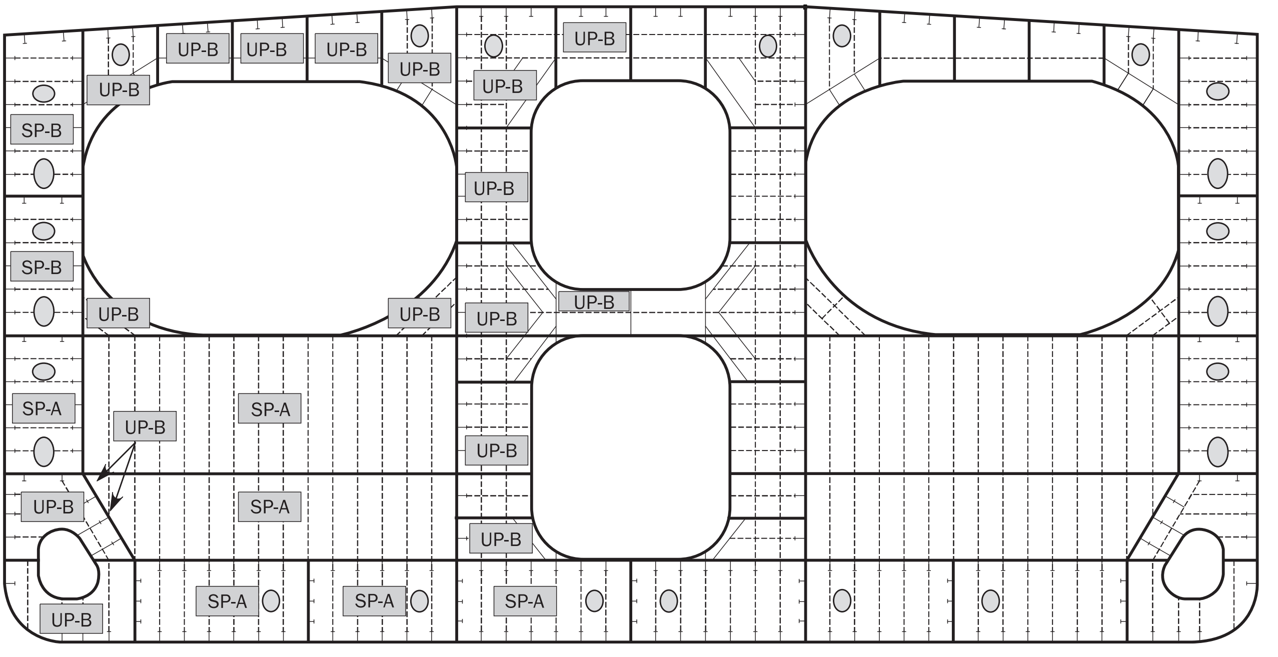


Figure 6 : Transverse bulkhead for offshore units

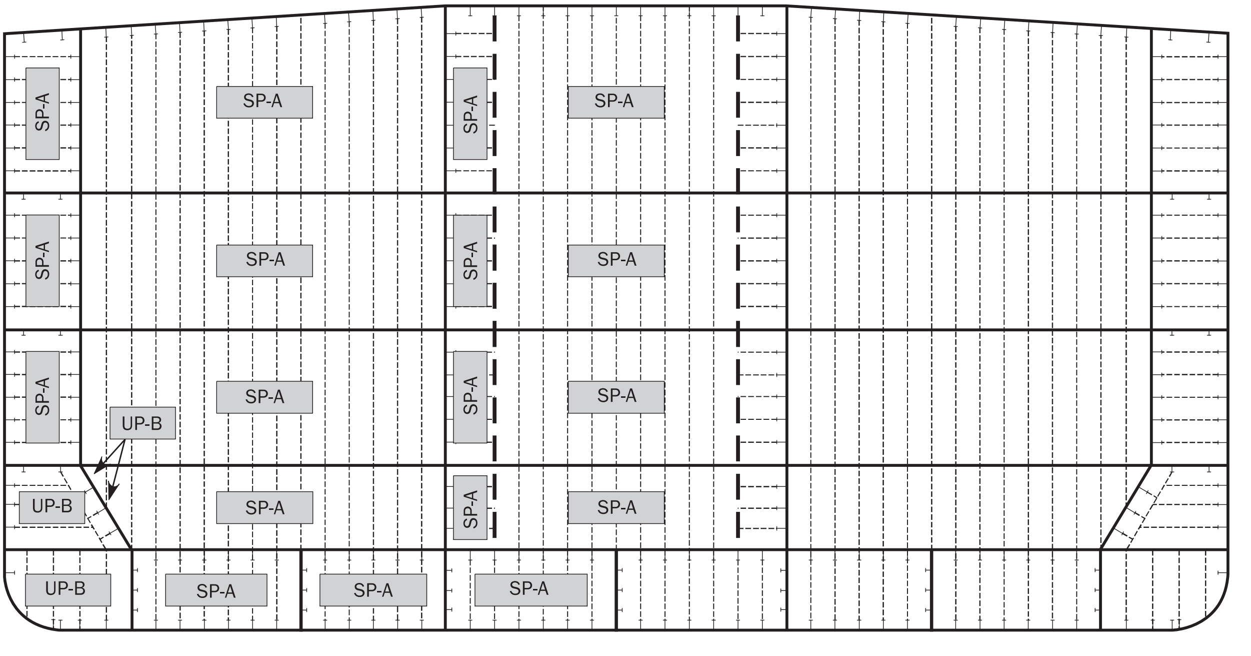


Figure 7 : Longitudinal plates for container ships

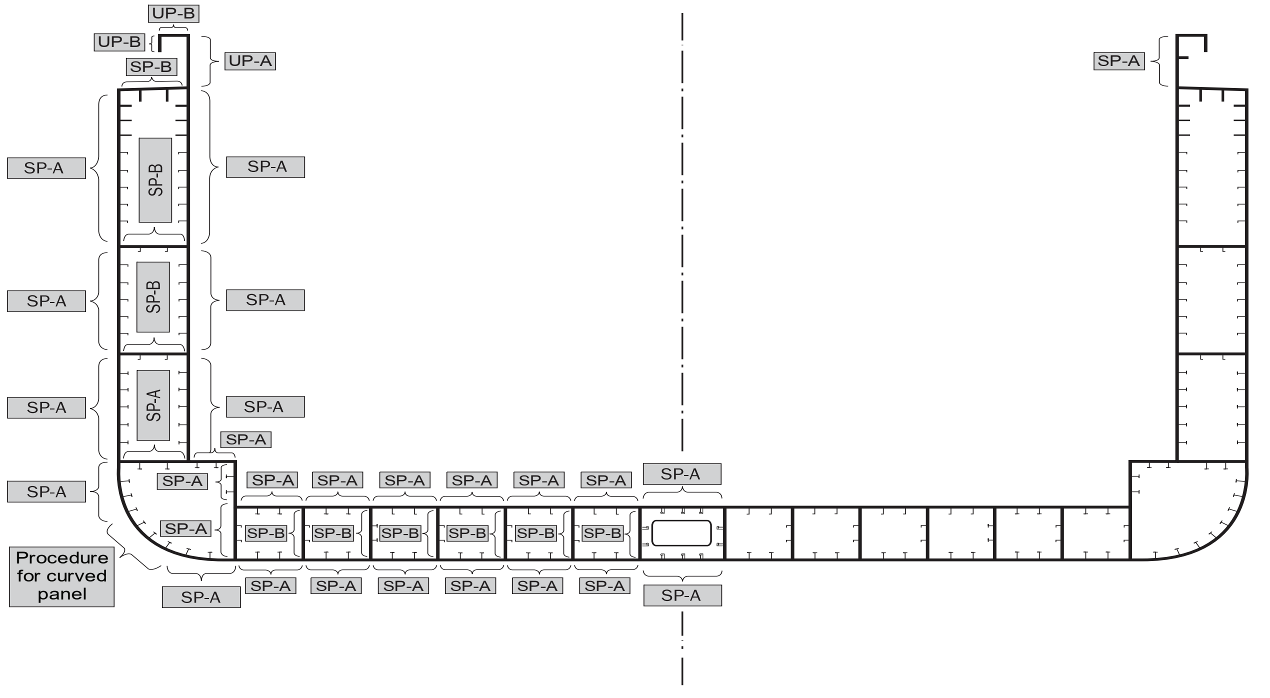


Figure 8 : Transverse web frames for container ships

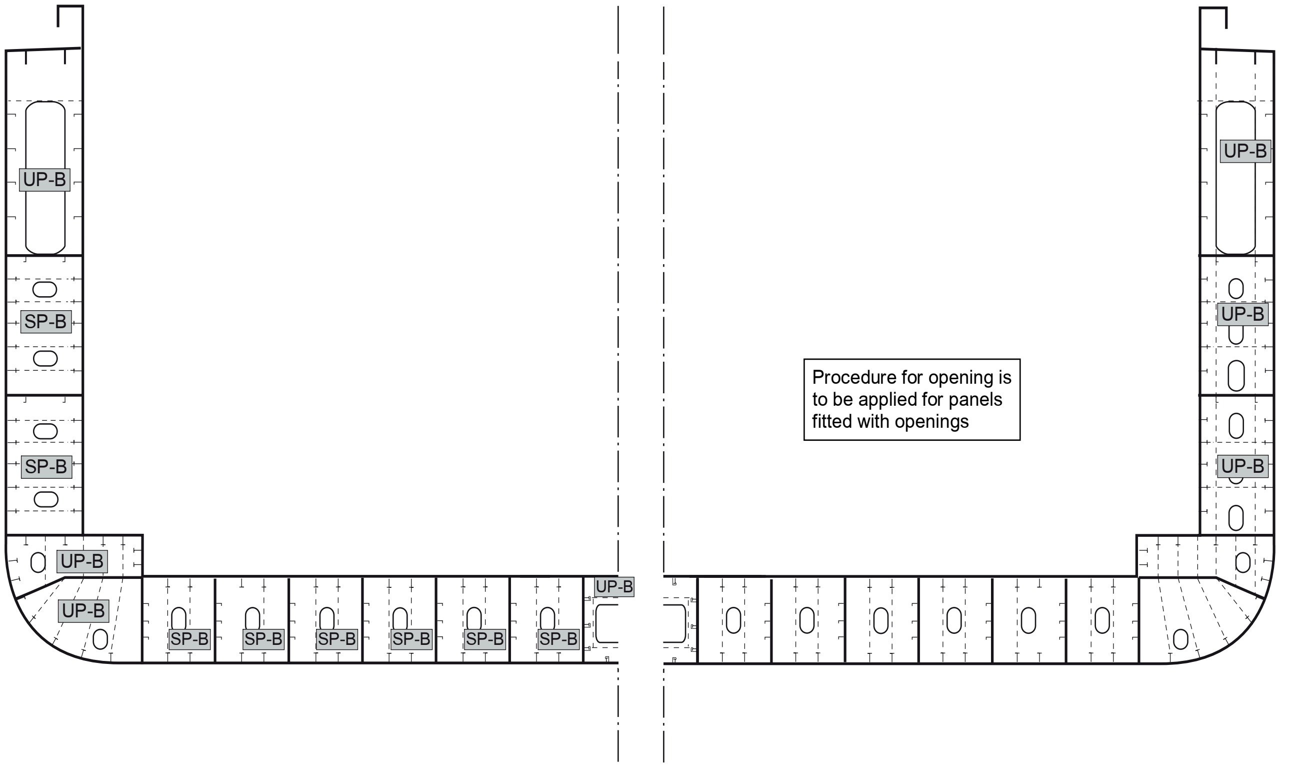


Figure 9 : Transverse bulkhead for container ships

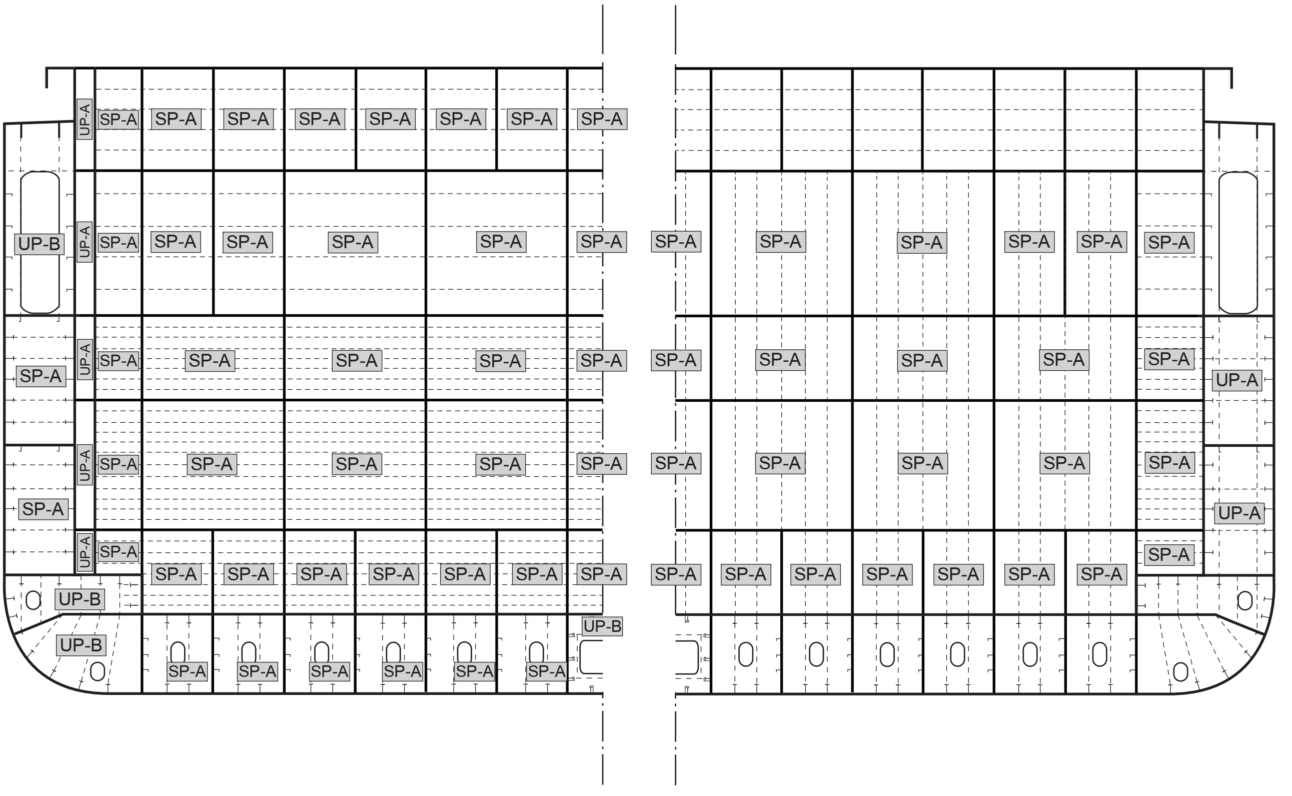


Figure 10 : Bulkhead internal members for container ships

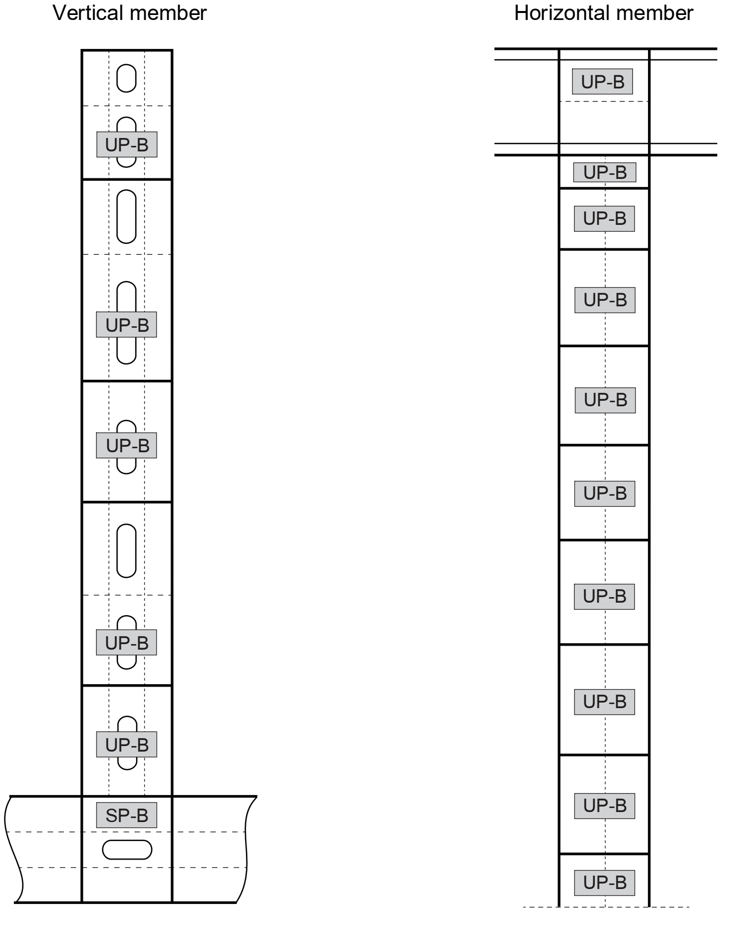
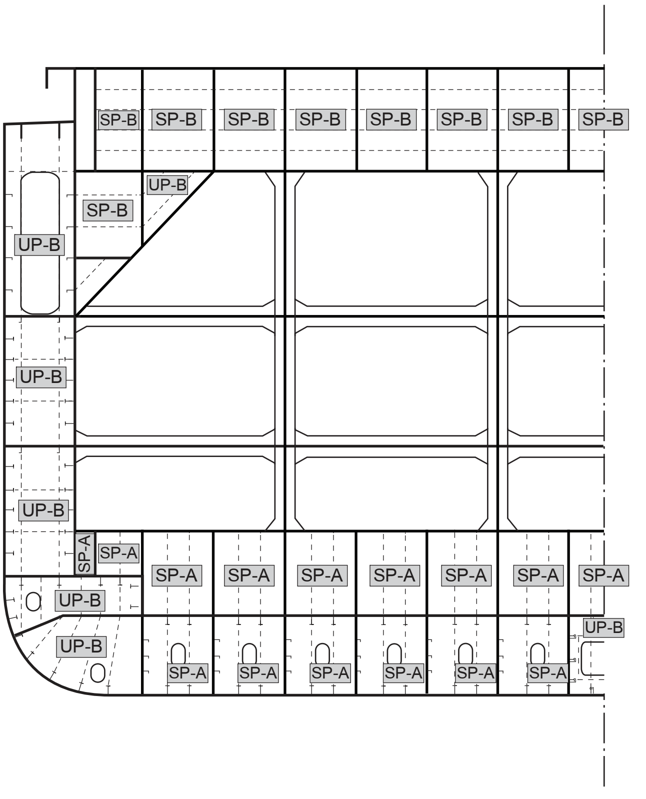


Figure 11 : Support bulkhead for container ships



Move Forward with Confidence

Marine & Offshore
Le Triangle de l’Arche - 8 Cours du Triangle - CS 50101
92937 Paris La Defense Cedex - France
Tel: + 33 (0)1 55 24 70 00
https://marine-offshore.bureauveritas.com/bv-rules