S35 (Feb 2023) Buckling Strength Assessment of Ship Structural Elements

TABLE OF CONTENTS

S35 (cont)

Note:

1. This UR is to be applied by IACS Societies to ships contracted for construction on or after 1 July 2024.

2. The “contracted for construction” date means the date on which the contract to build the vessel is signed between the prospective owner and the shipbuilder. For further details regarding the date of “contract for construction”, refer to IACS Procedural Requirement (PR) No. 29.

S35
(cont)

SECTION 1 APPLICATION AND DEFINITIONS

Abbreviations

EPP Elementary Plate Panel, as defined in [2.3.1].
PSM Primary Supporting Member.
SP Stiffened Panel, as defined in [2.3.3].
UP Unstiffened Panel, as defined in [2.3.3].
UR-S IACS Unified Requirements concerning Strength of Ships.

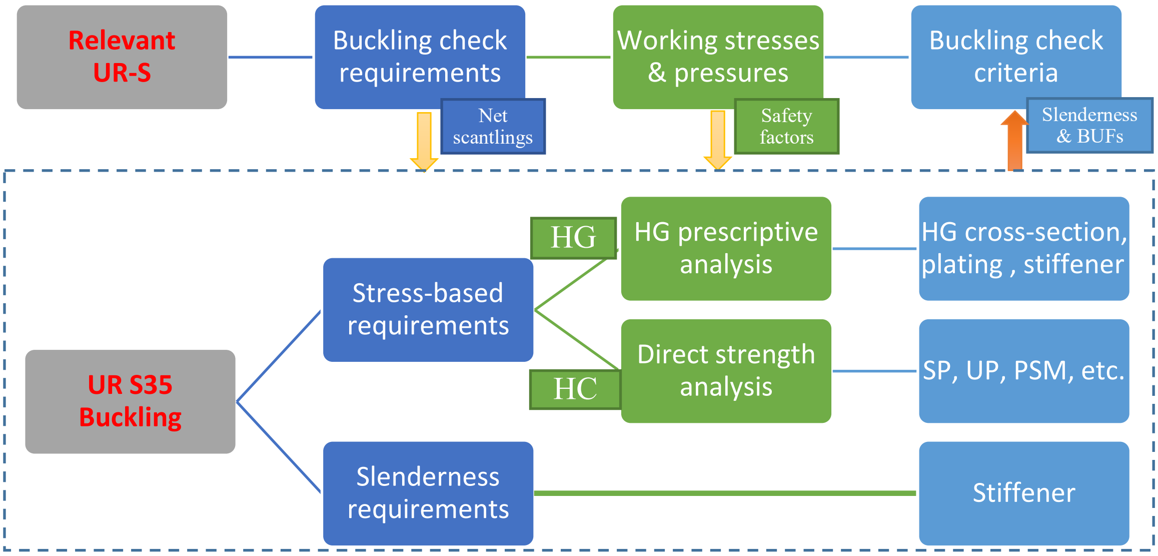
1. Application

1.1 General

1.1.1 Relevant Unified Requirements concerning Strength of Ships

This Unified Requirement (UR) establishes a general buckling assessment procedure as illustrated in Figure 1 and is to be applied in conjunction with UR S21 for hatch cover structures. UR S21 is referred to as Relevant UR-S hereafter in this UR.

Figure 1: Overview of applying this UR in conjunction with Relevant UR-S



Note: BUF stands for Buckling Utilisation Factor, HC stands for Hatch Cover, and HG stands for Hull Girder.

1.2 Application of this UR

1.2.1 Sections of this UR

The buckling checks are to be performed according to:

- Sec 1 for general definitions regarding buckling capacity, allowable buckling utilisation factors and buckling check criteria.

- Sec 2 for the slenderness requirements of longitudinal and transverse stiffeners.

- Sec 3 for the prescriptive buckling requirements of plates, longitudinal and transverse stiffeners, primary supporting members and other structures subject to hull girder stresses.

- Sec 4 for direct strength analysis (usually by finite element method) buckling requirements of hatch cover structural members including plates, stiffeners and primary supporting members.

- Sec 5 for the determination of buckling capacities of plate panels, stiffeners, primary supporting members and column structures.

1.2.2 Buckling assessment with this UR

For the buckling assessment of a ship hull girder, a hatch cover or some structural component, the slenderness requirements as defined in Sec 2 and the buckling requirements as defined in Sec 3 or Sec 4 are to be checked as per the requirements of the applicable Relevant UR-S.

1.2.3 Alternative methods

This UR contains the general methods for the determination of buckling capacities of plate panels, stiffeners, primary supporting members, and columns. For special cases not covered in this UR, such as a whole plate structure with stiffeners in two directions (i.e., a stiffened panel with both primary and secondary stiffeners), other more advanced methods, such as finite element analysis methods, can be used as deemed appropriate by the Society.

2 Terminology and Assumptions

2.1 Buckling

2.1.1 Buckling strength

Buckling strength or capacity refers to the strength of a structure under in-plane compressions and/or shear and lateral load. Buckling strength with consideration of the buckling behaviour in [2.1.2] gives a lower bound estimate of ultimate capacity, or the maximum load a structural member can carry without suffering major permanent set.
For each structural member, its buckling strength is to be taken as corresponding to the most unfavourable or critical buckling mode.

2.1.2 Buckling behaviour

Buckling strength assessment takes into account both elastic buckling and post-buckling behaviours. Post-buckling can consider the internal redistribution of loads depending on the load situation, slenderness and type of structure. Such as for the buckling assessment of plates, generally its positive elastic post-buckling effect can be utilized.

As such, for slender structures, the calculated buckling strength is typically higher than the ideal elastic buckling stress (minimum eigenvalue). Accepting elastic buckling of slender plate panels implies that large elastic deflections and reduced in-plane stiffness may occur at higher buckling utilisation levels.

S35 (cont)

2.2 Net Scantling Approach

2.2.1 General

Unless otherwise specified, all the scantling requirements, including slenderness requirements, in this UR are based on net scantlings obtained by removing full corrosion addition t_c from the gross offered thicknesses.

2.2.2 Corrosion addition

Corrosion addition t_c referred to in this UR is defined in the Relevant UR-S.

2.2.3 Stress calculation models

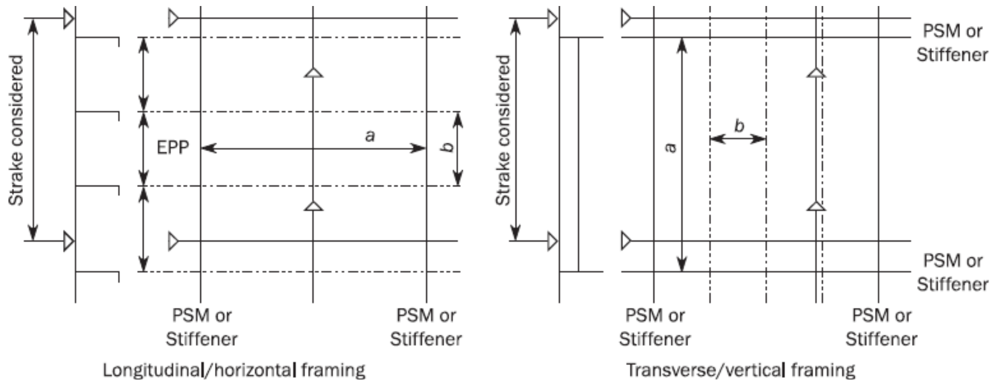
The structural models used for the calculation of stresses to be applied for buckling assessment, which are usually based on net scantlings, are defined in the Relevant UR-S.

2.3 Structural Idealisation

2.3.1 Elementary plate panel

An elementary plate panel (EPP) is the unstiffened part of the plating between stiffeners and/or primary supporting members. The plate panel length, *a*, and breadth, *b*, of the EPP are defined respectively as the longest and shortest plate edges, as shown in Figure 2.

Figure 2: Elementary plate panel (EPP) definition



2.3.2 Standard types of stiffeners

Definitions of the cross-sectional dimensions of typical stiffener types are shown in Figure 3, which are flat bars, bulb flats, angles, L2 and T bars. If applicable, other types of stiffeners can be idealized to one of the typical types in Figure 3 for buckling check. For the U-type stiffener which is usually fitted in some hatch covers, the definition of its cross-sectional dimensions is shown in Figure 4.

Unless otherwise specified, the full span or full length l, in mm, of a stiffener is to be used for buckling check, which equals to the spacing between primary supporting members.

S35
(cont)

Symbolic dimensions of the cross-sections are as below:

b_1 Width of the attached plate enclosed by the U-type stiffener, in mm, as shown in Figure 4.

b_2 Width of the attached plate between adjacent U-type stiffeners, in mm as shown in Figure 4.

b_f Width of the flange or face plate of the stiffener, in mm as shown in Figure 3 and Figure 4.

b_{f-out} Maximum distance, in mm, from mid thickness of the web to the flange edge, in mm, as shown in Figure 3.

d_f Breadth of the extended part of the flange for L2 profiles, in mm, as shown in Figure 3.

e_f Distance from attached plating to centre of flange, in mm, as shown in Figure 3. For its detailed definition, refer to Sec 5, Symbols.

h_w Depth of stiffener web, in mm, as shown in Figure 3 and Figure 4.

t_f Net flange thickness, in mm.

t_p Net thickness of plate, in mm.

t_w Net web thickness, in mm.

Figure 3: Dimensions of typical stiffener cross sections

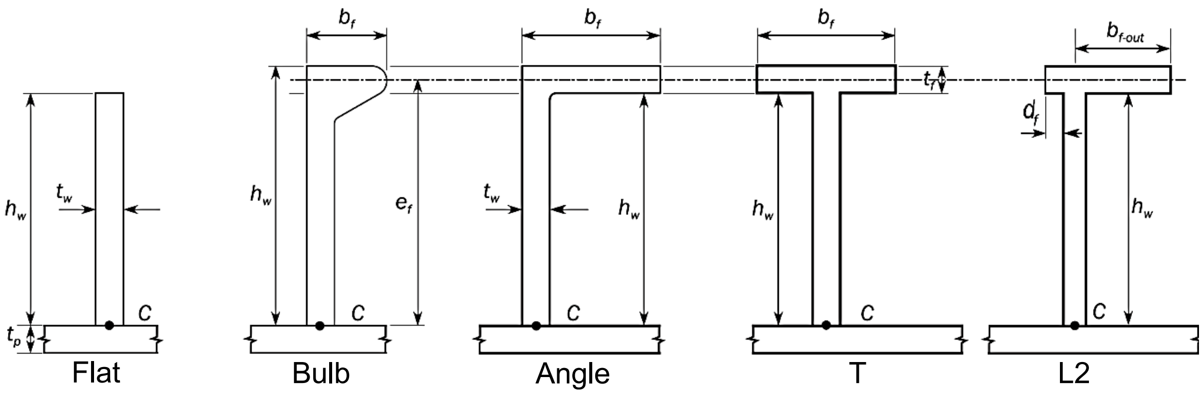
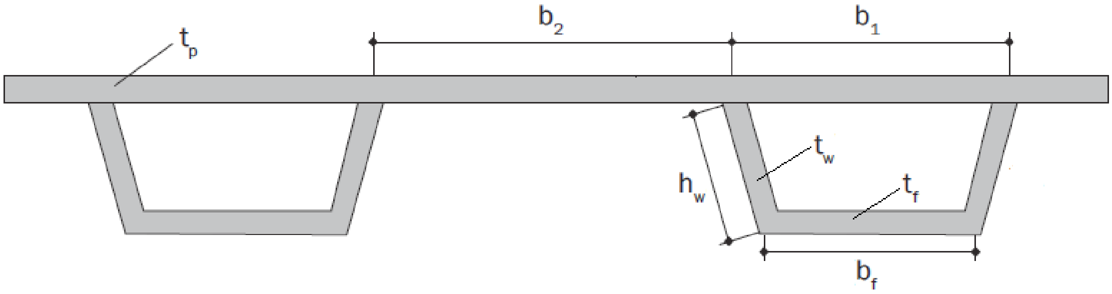


Figure 4: Dimensions of a U-type stiffener cross section



2.3.3 Stiffened panel (SP) and Unstiffened panel (UP)

For a panel with relatively strong interactive effect between the stiffener and its attached plate, each stiffener with its attached plate as a whole is to be modelled as a stiffened panel (SP), so as to be able to consider both of its local and global buckling modes.

However, for an EPP, if its buckling strength can be checked without considering its interactive effect with stiffeners fitted along its edges, it’s to be modelled as an unstiffened panel (UP).

2.4 Sign Convention

2.4.1 Stresses

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

3. Assessment Methods and Acceptance Criteria

3.1 Assessment Methods

3.1.1 Method A and Method B

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

- Method A: All the edges of the EPP are forced to remain straight (but free to move in the in-plane directions) due to the surrounding structure/neighbouring plates.

- Method B: The edges of the EPP are not forced to remain straight due to low in-plane stiffness at the edges and/or no surrounding structure/neighbouring plates.

3.1.2 SP-A, SP-B, UP-A and UP-B models

For the buckling assessment of the stiffened panel (SP) and unstiffened panel (UP) structural models defined in [2.3.3], with application of either Method A or Method B for the plate buckling assessment, the following four buckling assessment models are established:

- SP-A: a stiffened panel with application of Method A.
- SP-B: a stiffened panel with application of Method B.
- UP-A: an unstiffened panel with application of Method A.
- UP-B: an unstiffened panel with application of Method B.

3.2 Buckling Utilisation Factor

3.2.1 The utilisation factor, η , is defined as the ratio between the applied loads and the corresponding buckling capacity.

3.2.2 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 Figure 5, and is to be taken as:

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

S35
(cont)

where:

W_{act} Equivalent applied stress. The actual applied stresses are given in Sec 3 and Sec 4 respectively for buckling assessment by prescriptive and direct strength analysis.

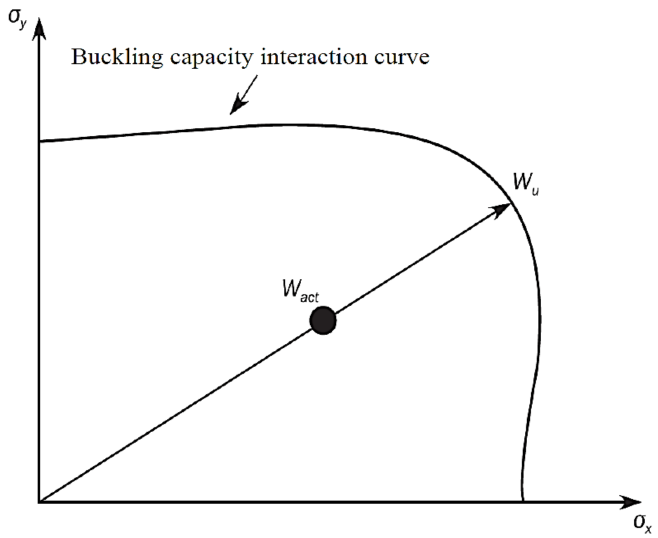
W_U Equivalent buckling capacity. For plates and stiffeners, their respective buckling or ultimate capacities are given in Sec 5.

γ_c Stress multiplier factor at failure.

For each typical failure mode, the corresponding buckling capacity of the panel is calculated by applying the actual stress combination and then increasing or decreasing the stresses proportionally until collapse occurs, i.e., when the increased or decreased stresses are on a buckling strength interaction curve or surface.

Figure 5 illustrates the buckling capacity and the buckling utilisation factor of a structural member subject to σ_x and σ_y stresses.

Figure 5: Illustration of buckling capacity and buckling utilisation factor



3.3 Allowable Buckling Utilisation Factor

3.3.1 The allowable buckling utilisation factor η_{all} is to be taken according to the Relevant UR-S.

3.4 Buckling Acceptance Criteria

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

$$\eta_{act} \leq \eta_{all}$$

where:

η_{act} Buckling utilisation factor based on the applied stress, defined in [3.2.2].

η_{all} Allowable buckling utilisation factor as defined in [3.3.1].

S35
(cont)

SECTION 2 SLENDERNESS REQUIREMENTS

Symbols

For symbols not defined in this section, refer to Sec 1, [2.3.2].

R_{eH} Specified minimum yield stress of the structural member being considered, in N/mm^2.

1. General

1.1 Introduction

1.1.1 The stiffener elements except for U-type stiffeners are to comply with the applicable slenderness and proportion requirements given in [2].

2. Stiffeners

2.1 Proportions of Stiffeners

2.1.1 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) Flange:

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

where:

C_w, C_f : Slenderness coefficients given in Table 1.

If requirement b) is not fulfilled, the effective free flange outstand, in mm, used in strength assessment including the calculation of actual net section modulus, is to be taken as:

$$b_{f-out-max} = C_f t_f \sqrt{\frac{235}{R_{eH}}}$$

Table 1: Slenderness coefficients

Type of Stiffener	C_w	C_f
Angle and L2 bars	75	12
T-bars	75	12
Bulb flats	45	-
Flat bars	22	-

S35 (cont)

For built-up profile where the relevant yielding strength for the web of built-up profile without the edge stiffener is acceptable, as an alternative the web can be assessed according to the web requirements of Angle and L2 bars in Table 1, and the edge stiffener can be assessed as a flat bar stiffener according to [2.1.1]. The requirement to flange in [2.1.2] shall still apply.

2.1.2 Net dimensions of angle and T-bars

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

$$b_f \geq 0.2h_w$$

3. Primary Supporting Members

3.1 Proportions and Stiffness

3.1.1 Proportions of web plate and flange

The scantlings of webs and flanges of primary supporting members are to comply with the Rules of the Classification Society.

S35
(cont)

SECTION 3 BUCKLING REQUIREMENTS FOR HULL GIRDER PRESCRIPTIVE ANALYSIS

Symbols

η_{all} Allowable buckling utilisation factor, as defined in Sec 1, [3.3.1].

LCP Load Calculation Point, as defined in [1.2.1].

1. General

1.1 Introduction

1.1.1 This section applies to plate panels including plane and curved plate panels, stiffeners and corrugation of longitudinal corrugated bulkheads subject to hull girder compression and shear stresses.

1.1.2 The ship longitudinal extent where the buckling check is performed for structural elements subject to hull girder stresses is to be in accordance with the Relevant UR-S.

1.1.3 Design load sets: The buckling check is to be performed for all design load sets corresponding to the design loading conditions defined in the Relevant UR-S with the most unfavourable pressure combinations.

For each design load set, for all static and dynamic load cases, the lateral pressure is to be determined at the load calculation point defined in [1.2.1], and is to be applied together with the hull girder stress combinations defined in the Relevant UR-S.

1.2 Definitions

1.2.1 Load calculation point

The load calculation points (LCP) for both elementary plate panels (EPP) and stiffeners are defined as follows:

a) LCP for hull girder stresses of EPP

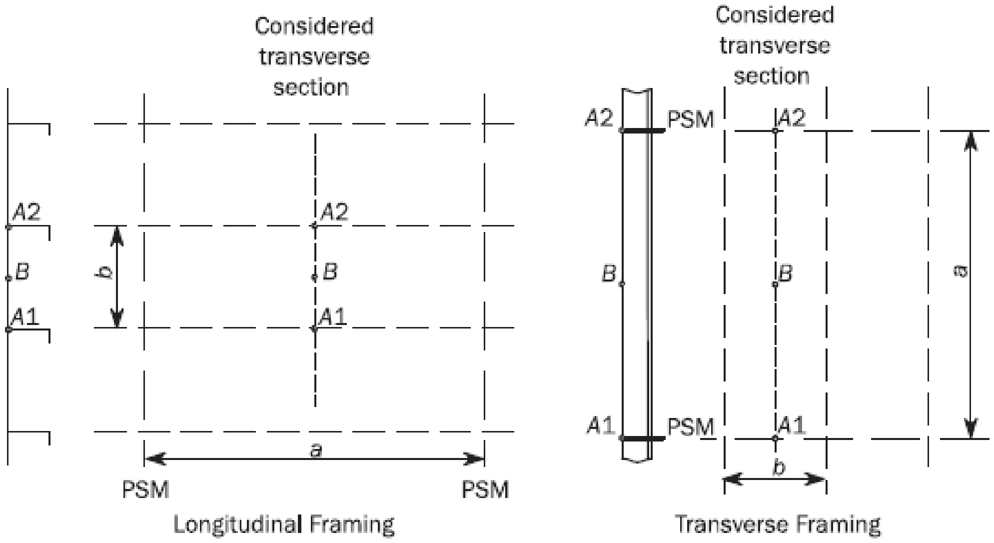
The hull girder stresses for EPP are to be calculated at the load calculation points defined in Table 1.

Table 1: Load calculation points (LCP) coordinates for plate buckling assessment

LCP coordinates	Hull girder bending stress		Hull girder shear stress
	Non horizontal plating	Horizontal plating	
x coordinate	Mid-length of the EPP		
y coordinate	Both upper and lower ends of the EPP (points A1 and A2 in Figure 1)	Outboard and inboard ends of the EPP (points A1 and A2 in Figure 1)	Mid-point of EPP (point B in Figure 1)
z coordinate	Corresponding to *x* and *y* values		

S35
(cont)

Figure 1: LCP for plate buckling assessment



b) LCP for hull girder stresses of longitudinal stiffeners

The hull girder stresses for longitudinal stiffeners are to be calculated at the following load calculation point:

- at the mid length of the considered stiffener.

- at the intersection point between the stiffener and its attached plate.

c) LCP for pressure of horizontal stiffeners

The load calculation point for the pressure is located at:

- Middle of the full length, l, of the considered stiffener.

- The intersection point between the stiffener and its attached plate.

d) LCP for pressure of non-horizontal stiffeners

The lateral pressure, P is to be calculated as the maximum between the value obtained at middle of the full length, l, and the value obtained from the following formulae:

$P = \frac{p_U + p_L}{2}$ when the upper end of the vertical stiffener is below the lowest zero pressure level.

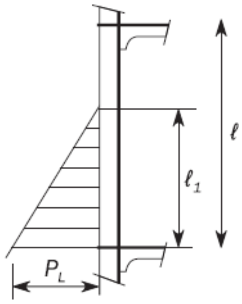
$P = \frac{l_1}{l}\frac{p_L}{2}$ when the upper end of the vertical stiffener is at or above the lowest zero pressure level, see Figure 2.

where:

l_1 Distance, in m, between the lower end of vertical stiffener and the lowest zero pressure level.

p_U, p_L Lateral pressures at the upper and lower end of the vertical stiffener span l, respectively.

Figure 2: Definition of pressure for vertical stiffeners



1.3 Assumptions for Equivalent Plate Panels

1.3.1 Longitudinal stiffening with varying plate thickness

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 of this equivalent plate panel, b_{eq}, in mm, is defined by the following formula:

$$b_{eq} = l_1 + l_2\left(\frac{t_1}{t_2}\right)^{1.5}$$

where:

l_1 Width of the part of the plate panel with the smaller plate thickness, t_1 , in mm, as defined in Figure 3.

l_2 Width of the part of the plate panel with the greater plate thickness, t_2 , in mm, as defined in Figure 3.

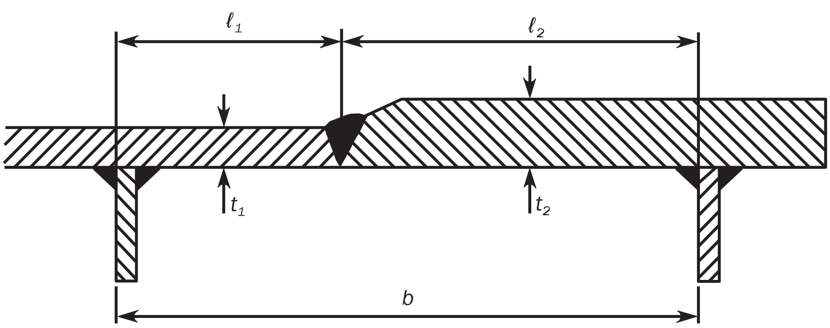
1.3.2 Transverse stiffening with varying plate thickness

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.

1.3.3 Plate panel with different materials

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

Figure 3: Plate thickness change over the width



S35
(cont)

2. Buckling Criteria

2.1 Overall Stiffened Panel

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

$$\eta_{overall} \leq \eta_{all}$$

where:

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

2.2 Plates

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

$$\eta_{plate} \leq \eta_{all}$$

where:

η_{plate} Maximum plate buckling utilisation factor as defined in Sec 5, [2.2] where SP-A model is to be used.

For the determination of η_{plate} of the vertically stiffened side shell plating of single side skin bulk carrier between hopper and topside tanks, the cases 12 and 16 of Sec 5, Table 3 corresponding to the shorter edge of the plate panel clamped are to be considered together with a mean σ_y stress and $\psi_y = 1$.

2.3 Stiffeners

2.3.1 The buckling strength of stiffeners or of side frames of single side skin bulk carriers is to satisfy the following criterion:

$$\eta_{stiffener} \leq \eta_{all}$$

where:

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

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

Note 2: The buckling check of the stiffeners is only applicable to the stiffeners fitted along the long edge of the buckling panel.

2.4 Vertically Corrugated Longitudinal Bulkheads

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

$$\eta_{shear} \leq \eta_{all}$$

where:

η_{shear} Maximum shear buckling utilisation factor, defined as

$$\eta_{shear} = \frac{\tau_{bhd}}{\tau_c}$$

S35
(cont)

τ_{bhd} Shear stress, in N/mm^2, in the bulkhead taken as the hull girder shear stress defined in the Relevant UR-S

τ_c Shear critical stress, in N/mm^2, as defined in Sec 5, [2.2.3].

2.5 Horizontally Corrugated Longitudinal Bulkheads

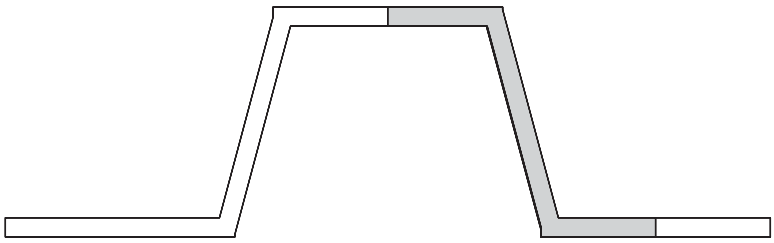
2.5.1 Each corrugation unit within the extension of half flange, web and half flange (i.e. single corrugation as shown in grey in Figure 4) is to satisfy the following criterion:

$$\eta_{column} \leq \eta_{all}$$

where:

η_{column} Overall column buckling utilisation factor, as defined in Sec 5, [3.1].

Figure 4: Single corrugation



S35 (cont)

SECTION 4 BUCKLING REQUIREMENTS FOR DIRECT STRENGTH ANALYSIS OF HATCH COVERS

Symbols

R_{eH_P} Yield stress of the plate panel, as defined in [2.1.3].

R_{eH_S} Yield stress of the stiffener, as defined in [2.1.3].

α Aspect ratio of the plate panel, as defined in the Symbol list of Sec 5.

η_{all} Allowable buckling utilisation factor, as defined in Sec 1, [3.3.1].

1. General

1.1 Introduction

1.1.1 The requirements of this Section apply to the buckling assessment of hatch cover structural members based on direct strength analysis (usually by finite element method) and subjected to normal stress, shear stress and lateral pressure.

1.1.2 All structural elements in the direct strength analysis carried out according to the Relevant UR-S are to be assessed individually. The buckling checks are to be performed for the following structural elements:

- Stiffened and unstiffened panels.
- Web plate in way of openings.

2. Stiffened and Unstiffened Panels

2.1 General

2.1.1 The plate panel of a hatch cover structure is to be modelled as stiffened panel (SP) or unstiffened panel (UP), with either Method A or Method B as defined in Sec 1, [3.1.1] to be used for the calculation of the plate buckling capacity, which in combination is also equivalent to use the buckling assessment models defined in Sec 1, [3.1.2].

2.1.2 Average thickness of plate panel

For FE analysis, where the plate thickness along a plate panel is not constant, the panel used for the buckling assessment is to be modelled with a weighted average thickness 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.

t_i : Net thickness of the *i*-th plate element.

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

S35
(cont)

2.1.3 Yield stress of the plate panel and stiffener

The panel yield stress R_{eH_P} is taken as the minimum value of the specified yield stresses of the elements within the plate panel.

The stiffener yield stress R_{eH_S} is taken as the minimum value of the specified yield stresses of the elements within the stiffener.

2.2 Stiffened Panels

2.2.1 For a stiffened panel (SP), each stiffener with attached plate is to be idealized as a stiffened panel model of the extent defined in the Relevant UR-S.

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

2.3.1 The buckling check of the stiffeners of stiffened panels is only applicable to the stiffeners fitted along the longer side edges of the buckling panel.

2.3 Unstiffened Panels

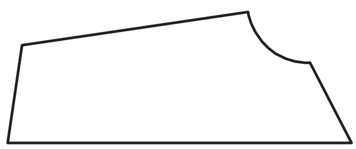
2.3.1 Irregular plate panel

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

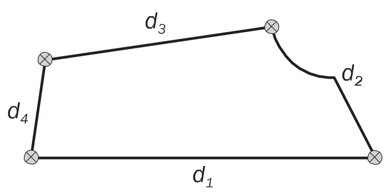
2.3.2 Equivalent EPP of an unstiffened panel with irregular geometry

Unstiffened panels with irregular geometry are to be idealised to equivalent 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.

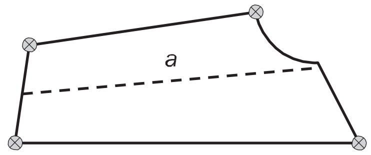


b) The distances along the plate bounding polygon between the corners are calculated, i.e. the sum of all the straight-line segments between the end points.



S35
(cont)

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



d) A line joins the middle points of the chosen opposite edges (i.e. a mid-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 of the capacity model, a, measured from one end point.

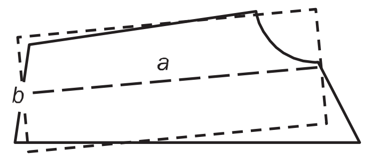
e) The length of shorter side, b, in mm, is to be taken as:

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

where:

A : Area of the plate, in mm²

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

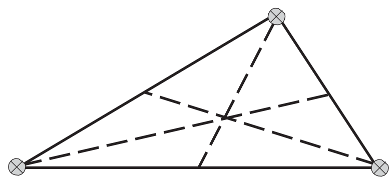


f) The stresses from the direct strength analysis are to be transformed into the local coordinate system of the equivalent rectangular panel. These stresses are to be used for the buckling assessment.

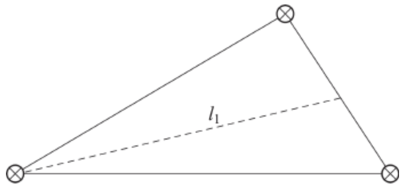
2.3.3 Modelling of an unstiffened plate panel with triangular geometry

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

a) Medians are constructed as shown below.



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



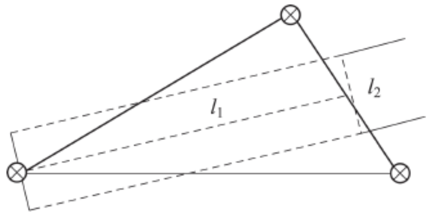
S35
(cont)

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

$$l_2 = \frac{A}{l_1}$$

where:

A Area of the plate, in mm²



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

$$b = \frac{l_2}{C_{tri}}$$

$$a = l_1 C_{tri}$$

where:

$$C_{tri} = 0.4\frac{l_2}{l_1} + 0.6$$

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

2.4 Reference Stress

2.4.1 The stress distribution is to be taken from the direct strength analysis according to the Relevant UR-S and applied to the buckling model.

2.4.2 For FE analysis, the reference stresses are to be calculated using the stress-based reference stresses as defined in Appendix 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.

For FE analysis, where the lateral pressure is not constant over a buckling panel defined by a number of finite plate elements, an average lateral pressure, N/mm^2, 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 mm^2.

P_i : Lateral pressure of the i-th plate element, in N/mm^2.

n : Number of finite elements in the buckling panel.

S35 (cont)

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} Plate buckling utilisation factor, equal to η_{plate} as defined in Sec 5, [2.2] where UP-A model is to be used.

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} Plate buckling utilisation factor, equal to η_{plate} as defined in Sec 5, [2.2] where UP-B model is to be used.

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} Buckling utilisation factor of the stiffened panel, taken as the maximum of the buckling utilisation factors calculated as below:

- The overall stiffened panel buckling utilisation factor $\eta_{overall}$ as defined in Sec 5, [2.1].
- The plate buckling utilisation factor η_{plate} as defined in Sec 5, [2.2] where SP-A model is to be used.
- The stiffener buckling utilisation factor $\eta_{stiffener}$ 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 strength check can only be fulfilled when the overall stiffened panel capacity check, as defined in Sec 5, [2.1], is satisfied.

S35 (cont)

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} Buckling utilisation factor of the stiffened panel, taken as the maximum of the buckling utilisation factors calculated as below:

- The overall stiffened panel buckling utilisation factor $\eta_{overall}$ as defined in Sec 5, [2.1].
- The plate buckling utilisation factor η_{plate} as defined in Sec 5, [2.2] where SP-B model is to be used.
- The stiffener buckling utilisation factor $\eta_{stiffener}$ 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 strength check can only be fulfilled when the overall stiffened panel capacity check, 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 with openings is to satisfy the following criterion:

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

where:

$\eta_{opening}$ Maximum web plate utilisation factor in way of openings, calculated with the definition in Sec 1, [3.2.2] and the stress multiplier factor at failure γ_c which can be calculated following the requirements in Sec 5, [2.4].

S35
(cont)

SECTION 5 BUCKLING CAPACITY

Symbols

A_p Net sectional area of the stiffener attached plating, in mm^2, taken as:

$$A_p = st_p$$

A_s Net sectional area of the stiffener without attached plating, in mm^2.

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:

- For $\sigma_x > 0$
  - For prescriptive assessment:

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

  - For FE analysis:

    $$b_{eff1} = C_x b$$

- For $\sigma_x \leq 0$

  $$b_{eff1} = b$$

b_f Breadth of the stiffener flange, in mm.

b_1, b_2 Width of plate panel on each side of the considered stiffener, in mm. For stiffened panels fitted with U-type stiffeners, b_1 and b_2 are as defined in Sec 1, Figure 4.

C_{x1}, C_{x2} Reduction factor defined in Table 3 calculated for the EPP1 and EPP2 on each side of the considered stiffener according to case 1.

d Length of the side parallel to the cylindrical axis of the cylinder corresponding to the curved plate panel as shown in Table 4, in mm.

d_f Breadth of the extended part of the flange for L2 profiles, in mm, as shown in Sec 1, Figure 3.

e_f Distance from attached plating to centre of flange, in mm, as shown in Sec 1, Figure 3 to be taken as:

$e_f = h_w$ for flat bar profile.

$e_f = h_w - 0.5t_f$ for bulb profile.

$e_f = h_w + 0.5t_f$ for angle, L2 and T profiles.

F_{long} Coefficient defined in [2.2.4].

F_{tran} Coefficient defined in [2.2.5].

h_w Depth of stiffener web, in mm, as shown in Sec 1, Figure 3.

S35
(cont)

l	Span, in mm, of stiffener equal to spacing between primary supporting members or span of side frame equal to the distance between the hopper tank and top wing tank in way of the side shell.
R	Radius of curved plate panel, in mm.
R_{eH_P}	Specified minimum yield stress of the plate in N/mm^2.
R_{eH_S}	Specified minimum yield stress of the stiffener in N/mm^2.
S	Partial safety factor, unless otherwise specified in the Relevant UR-S, to be taken as 1.0.
t_p	Net thickness of plate panel, in mm.
t_w	Net stiffener web thickness, in mm.
t_f	Net flange thickness, in mm.
x-axis	Local axis of a rectangular buckling panel parallel to its long edge.
y-axis	Local axis of a rectangular buckling panel perpendicular to its long edge.
α	Aspect ratio of the plate panel, defined in Table 3 to be taken as: $\alpha = \frac{a}{b}$
β	Coefficient taken as: $\beta = \frac{1-\psi}{\alpha}$
ω	Coefficient taken as: $\omega = min(3, \alpha)$
σ_x	Normal stress applied on the edge along x-axis of the buckling panel, in N/mm^2.
σ_y	Normal stress applied on the edge along y-axis of the buckling panel, in N/mm^2.
σ_1	Maximum normal stress along a panel edge, in N/mm^2.
σ_2	Minimum normal stress along a panel edge, in N/mm^2.
σ_E	Elastic buckling reference stress, in N/mm^2 to be taken as:

- For the application of the limit state of plane plate panels 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 the limit state of curved plate panels according to [2.2.6]:

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

τ	Applied shear stress, in N/mm^2.
τ_c	Buckling strength in shear, in N/mm^2, as defined in [2.2.3].
ψ	Edge stress ratio to be taken as: $\psi = \frac{\sigma_2}{\sigma_1}$
γ	Stress multiplier factor acting on loads. When the factor is such that the loads reach the interaction formulae, $\gamma = \gamma_c$.
γ_c	Stress multiplier factor at failure.
γ_{GEB}	Stress multiplier factor of global elastic buckling capacity.

S35 (cont)

1. General

1.1 Introduction

1.1.1 This section contains the methods for determination of the buckling capacities of plate panels, stiffeners, primary supporting members and columns.

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 hull girder prescriptive buckling requirements.
- Sec 4 for direct strength analysis buckling requirements of hatch covers.

1.1.3 Buckling capacity

The 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, respectively.

1.1.4 Buckling utilisation factor

The buckling utilisation factor 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 applied and considered as constant for the calculation of buckling capacities as defined in [1.1.3].

2. Buckling Capacity of Plate Panels

2.1 Overall Stiffened Panels

2.1.1 The elastic stiffened panel limit state is based on the following interaction formula, which sets a precondition for the buckling check of stiffeners in accordance with [2.3.4]:

$$\frac{\gamma_c}{\gamma_{GEB}} = 1$$

with the corresponding buckling utilization factor defined as

$$\eta_{overall} = \frac{1}{\gamma_c}$$

where the stress multiplier factors of global elastic buckling capacity, γ_{GEB}, are to be calculated based on the following formulae:

$$\gamma_{GEB} = \gamma_{GEB,bi+\tau} \quad \text{for } \tau \neq 0 \text{ and } (\sigma_x > 0 \text{ or } \sigma_y > 0)$$

$$\gamma_{GEB} = \gamma_{GEB,bi} \quad \text{for } \tau = 0 \text{ and } (\sigma_x > 0 \text{ or } \sigma_y > 0)$$

$$\gamma_{GEB} = \gamma_{GEB,\tau} \quad \text{for } \tau \neq 0 \text{ and } (\sigma_x \leq 0 \text{ and } \sigma_y \leq 0)$$

S35 (cont)

where $\gamma_{GEB,bi+\tau}$, $\gamma_{GEB,bi}$ and $\gamma_{GEB,\tau}$ are stress multiplier factors of the global elastic buckling capacity for different load combinations as defined in [2.1.2], [2.1.3] and [2.1.4], respectively. For the calculation of $\gamma_{GEB,bi+\tau}$, $\gamma_{GEB,bi}$ and $\gamma_{GEB,\tau}$, neither σ_x nor σ_y shall be taken less than 0.

σ_x, σ_y Applied normal stress to the plate panel, in N/mm^2, to be taken as defined in [2.2.7].

τ Applied shear stress, in N/mm^2, to be taken as defined in [2.2.7].

2.1.2 The stress multiplier factor $\gamma_{GEB,bi}$ for the stiffened panel subjected to biaxial loads is taken as:

$$\gamma_{GEB,bi} = \frac{\pi^2}{L_{B1}^2 L_{B2}^2} \frac{[D_{11}L_{B2}^4 + 2(D_{12} + D_{33})n^2 L_{B1}^2 L_{B2}^2 + n^4 D_{22} L_{B1}^4]}{L_{B2}^2 N_x + n^2 L_{B1}^2 N_y}$$

where:

N_x Load per unit length applied on the edge along x-axis of the stiffened panel, in N/mm, taken as

$$N_x = \sigma_{x,av}(A_p + A_s)/s$$

For stiffened panels fitted with U-type stiffeners, stiffener spacing s is taken as:

$$s = b_1 + b_2$$

where b_1 and b_2 are as defined in Sec 1, Figure 4.

N_y Load per unit length applied on the edge along y axis of the stiffened panel, in N/mm, taken as $N_y = c\sigma_y t_p$

L_{B1} Stiffener span, in mm, distance between primary supporting members, i.e. $L_{B1} = l$. Specially, for vertically stiffened side shell of single side skin bulk carriers, $L_{B1} = 0.8l$.

L_{B2} Total width of stiffened panel between lateral supports, in mm, taken as 6 times of the stiffener spacing, i.e. 6*s*.

n Number of half waves along the direction perpendicular to the stiffener axis. The factor $\gamma_{GEB,bi}$ is to be minimized with respect to the wave parameters n, i.e. to be taken as the smallest value larger than zero.

c Factor taking into account the normal stress distribution in the attached plating acting perpendicular to the stiffener's axis:

$$c = 0.5(1 + \psi) \quad \text{for } 0 \le \psi \le 1$$

$$c = \frac{1}{2(1 - \psi)} \quad \text{for } \psi < 0$$

ψ Edge stress ratio for case 2 according to Table 3.

$\sigma_{x,av}$ Average stress for both plate and stiffener, taken as:

$$\sigma_{x,av} = \sigma_x - \nu c \sigma_y A_s/(A_p + A_s) \ge 0 \quad \text{for } \sigma_x > 0 \; and \; \sigma_y > 0$$

$$\sigma_{x,av} = \sigma_x \quad \text{for } \sigma_x \le 0 \text{ or } \sigma_y \le 0$$

S35 (cont)

$D_{11}, D_{12}, D_{22}, D_{33}$ Bending stiffness coefficients, in Nmm, of the stiffened panel, defined in general as:

$$\left.\begin{aligned} D_{11} &= \frac{EI_{eff}10^4}{s} \\ D_{12} &= \frac{Et_p^3\nu}{12(1-\nu^2)} \\ D_{22} &= \frac{Et_p^3}{12(1-\nu^2)} \\ D_{33} &= \frac{Et_p^3}{12(1+\nu)} \end{aligned}\right\}$$

For stiffened panels fitted with U-type stiffeners, D_{12} and D_{22} are defined as:

$$D_{22} = \frac{Et_p^3}{12(1-\nu^2)}\left[1.2 + 4.8 \times Min\left(1.0, \frac{b_1^2}{h_w(b_1+b_2)}\right) \times Min\left(1.0, \left(\frac{t_w}{t_p}\right)^3\right)\right]$$

$$D_{12} = \nu D_{22}$$

h_w is the breadth of U-type stiffener web as defined in Sec 1, Figure 4.

I_{eff} Moment of inertia, in cm^4, of the stiffener including the effective width of the attached plating, same as I defined in [2.3.4].

2.1.3 The stress multiplier factor $\gamma_{GEB,\tau}$ for the stiffened panel subjected to pure shear load is taken as:

$$\gamma_{GEB,\tau} = \frac{\sqrt[4]{D_{11}^3 D_{22}}}{(L_{B1}/2)^2 N_{xy}}\left[8.125 + 5.64\sqrt{\frac{(D_{12}+D_{33})^2}{D_{11}D_{22}}} - 0.6\frac{(D_{12}+D_{33})^2}{D_{11}D_{22}}\right] \ for\ D_{11}D_{22} \geq (D_{12}+D_{33})^2$$

$$\gamma_{GEB,\tau} = \frac{\sqrt{2D_{11}(D_{12}+D_{33})}}{(L_{B1}/2)^2 N_{xy}}\left[8.3 + 1.525\frac{D_{11}D_{22}}{(D_{12}+D_{33})^2} - 0.493\frac{D_{11}^2 D_{22}^2}{(D_{12}+D_{33})^4}\right] \ for\ D_{11}D_{22} < (D_{12}+D_{33})^2$$

where:

$$N_{xy} = \tau t_p$$

2.1.4 The stress multiplier factor $\gamma_{GEB,bi+\tau}$ for the stiffened panel subjected to combined loads is taken as:

$$\gamma_{GEB,bi+\tau} = \frac{1}{2}\gamma_{GEB,\tau}{}^2\left[-\frac{1}{\gamma_{GEB,bi}} + \sqrt{\frac{1}{\gamma_{GEB,bi}{}^2} + 4\frac{1}{\gamma_{GEB,\tau}{}^2}}\right]$$

where $\gamma_{GEB,bi}$ and $\gamma_{GEB,\tau}$ are as defined in [2.1.2] and [2.1.3], respectively.

S35 (cont)

2.2 Plates

2.2.1 Plate limit state

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

$$\left(\frac{\gamma_{c1}\sigma_x S}{\sigma_{cx}}\right)^{e_0} - B\left(\frac{\gamma_{c1}\sigma_x S}{\sigma_{cx}}\right)^{\frac{e_0}{2}}\left(\frac{\gamma_{c1}\sigma_y S}{\sigma_{cy}}\right)^{\frac{e_0}{2}} + \left(\frac{\gamma_{c1}\sigma_y S}{\sigma_{cy}}\right)^{e_0} + \left(\frac{\gamma_{c1}|\tau|S}{\tau_c}\right)^{e_0} = 1$$

$$\left(\frac{\gamma_{c2}\sigma_x S}{\sigma_{cx}}\right)^{\frac{2}{\beta_p^{0.25}}} + \left(\frac{\gamma_{c2}|\tau|S}{\tau_c}\right)^{\frac{2}{\beta_p^{0.25}}} = 1 \ for\ \sigma_x \geq 0$$

$$\left(\frac{\gamma_{c3}\sigma_y S}{\sigma_{cy}}\right)^{\frac{2}{\beta_p^{0.25}}} + \left(\frac{\gamma_{c3}|\tau|S}{\tau_c}\right)^{\frac{2}{\beta_p^{0.25}}} = 1 \ for\ \sigma_y \geq 0$$

$$\frac{\gamma_{c4}|\tau|S}{\tau_c} = 1$$

with

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

and the corresponding buckling utilization factor defined as

$$\eta_{plate} = \frac{1}{\gamma_c}$$

where:

σ_x, σ_y Applied normal stress to the plate panel, in N/mm^2, to be taken as defined in [2.2.7].

τ Applied shear stress to the plate panel, in N/mm^2.

σ_{cx} Ultimate buckling stress, in N/mm^2, in direction parallel to the longer edge of the buckling panel as defined in [2.2.3].

σ_{cy} Ultimate buckling stress, in N/mm^2, in direction parallel to the shorter edge of the buckling panel as defined in [2.2.3].

τ_c Ultimate buckling shear stress, in N/mm^2, as defined in [2.2.3].

$\gamma_{c1}, \gamma_{c2}, \gamma_{c3}, \gamma_{c4}$: Stress multiplier factors at failure for each of the above different limit states. γ_{c2} and γ_{c3} are only to be considered when $\sigma_x \geq 0$ and $\sigma_y \geq 0$ respectively.

B Coefficient given in Table 1

e_0 Coefficient given in Table 1

β_p Plate slenderness parameter taken as:

$$\beta_p = \frac{b}{t_p}\sqrt{\frac{R_{eH\_P}}{E}}$$

Table 1: Definition of coefficients B and e_0

Applied stress	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

S35
(cont)

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 Table 3 and Table 4.

2.2.3 Ultimate buckling stresses

The ultimate buckling stresses of plate panels, in N/mm^2, 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/mm^2, 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 Table 3

- For the 1st Equation of [2.2.1], when $\sigma_x < 0$ or $\sigma_y < 0$, the reduction factors are to be taken as:

$$C_x = C_y = C_\tau = 1.$$

- For other cases:
  - For SP-A and UP-A, C_y is calculated according to Table 3 by using

$$c_1 = \left(1 - \frac{1}{\alpha}\right) \geq 0$$

  - For SP-B and UP-B, C_y is calculated according to Table 3 by using

$$c_1 = 1$$

  - For vertically stiffened single side skin of bulk carrier, C_y is calculated according to Table 3 by using

$$c_1 = \left(1 - \frac{1}{\alpha}\right) \geq 0$$

  - For corrugation of corrugated bulkheads, C_y is calculated according to Table 3 by using

$$c_1 = \left(1 - \frac{1}{\alpha}\right) \geq 0$$

The boundary conditions for plates are to be considered as simply supported, see cases 1, 2 and 15 of Table 3. If the boundary conditions differ significantly from simple support, a more appropriate boundary condition can be applied according to the different cases of Table 3 subject to the agreement of the Society.

S35 (cont)

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 Table 2. An average value of F_{long} is to be used for plate panels having different edge stiffeners. For stiffener types other than those mentioned in Table 2, the value of c is to be agreed by the Society. In such a case, value of c higher than those mentioned in Table 2 can be used, provided it is verified by buckling strength check of panel using non-linear FE analysis and deemed appropriate by the Society.

Table 2: Correction factor F_{long}

<table>
<tr><th colspan="3">Structural element types</th><th>F_{long}</th><th>c</th></tr>
<tr><td colspan="3">Unstiffened Panel</td><td>1.0</td><td>N/A</td></tr>
<tr><td rowspan="7">Stiffened Panel</td><td colspan="2">Stiffener not fixed at both ends</td><td>1.0</td><td>N/A</td></tr>
<tr><td rowspan="6">Stiffener fixed at both ends</td><td>Flat bar (1)</td><td rowspan="4">$F_{long} = c + 1 \ for \ \frac{t_w}{t_p} > 1$
$F_{long} = c\left(\frac{t_w}{t_p}\right)^3 + 1 \ for \ \frac{t_w}{t_p} \le 1$</td><td>0.10</td></tr>
<tr><td>Bulb profile</td><td>0.30</td></tr>
<tr><td>Angle and L2 profiles</td><td>0.40</td></tr>
<tr><td>T profile</td><td>0.30</td></tr>
<tr><td>Girder of high rigidity (e.g. bottom transverse)</td><td>1.4</td><td>N/A</td></tr>
<tr><td>U-type profile fitted on hatch cover(2)</td><td>- Plate on which the U-type profile is fitted, including EPP b_1 and EPP b_2
- For $b_2 < b_1$: $F_{long} = 1$
- For $b_2 \ge b_1$:
$F_{long} = \left(1.55 - 0.55\frac{b_1}{b_2}\right)\left[1 + c\left(\frac{t_w}{t_p}\right)^3\right]$
- Other plates of the U-type profile: $F_{long} = 1$</td><td>0.2</td></tr>
<tr><td colspan="5">(1) t_w is the net web thickness, in mm, without the correction defined in [2.3.2].
(2) b_1, b_2 and t_w are defined in Sec.1, Figure 4.</td></tr>
</table>

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 bulk carrier, between the hopper and top wing tank:
  - $F_{tran} = 1.25$ when the two adjacent frames are supported by one tripping bracket fitted in way of the adjacent plate panels.
  - $F_{tran} = 1.33$ when the two adjacent frames are supported by two tripping brackets each fitted in way of the adjacent plate panels.
  - $F_{tran} = 1.15$ elsewhere.

S35 (cont)

- For the attached plate of a U-type stiffener fitted on a hatch cover:

$F_{tran} = Max(3 - 0.08(F_{tran0} - 6)^2, 1.0) \leq 2.25$

where:

$$F_{tran0} = Min\left(\frac{b_2}{b_1} + \frac{6b_2^2}{\pi^2 h_w (b_1 + b_2)}\left(\frac{t_w}{t_p}\right)^3, 6\right) \text{ for EPP } b_2$$

$$F_{tran0} = Min\left(\frac{b_1}{b_2} + \frac{6b_1^2}{\pi^2 h_w (b_2 + b_1)}\left(\frac{t_w}{t_p}\right)^3, 6\right) \text{ for EPP } b_1$$

with b_1, b_2 and h_w as defined in Sec.1, Figure 4.

Coefficient F defined in Case 2 of Table 3 is to be replaced by the following formula:

$$F = \left[1 - \left(\frac{K_y}{0.91F_{tran}} - 1\right)/\lambda_p^2\right] c_1 \geq 0$$

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

2.2.6 Curved plate panels

This requirement for curved plate limit state is applicable when $R/t_p \leq 2500$. 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:

$$\left(\frac{\gamma_c \sigma_{ax} S}{C_{ax} R_{eH\_P}}\right)^{1.25} - 0.5\left(\frac{\gamma_c \sigma_{ax} S}{C_{ax} R_{eH\_P}}\right)\left(\frac{\gamma_c \sigma_{tg} S}{C_{tg} R_{eH\_P}}\right) + \left(\frac{\gamma_c \sigma_{tg} S}{C_{tg} R_{eH\_P}}\right)^{1.25} + \left(\frac{\gamma_c \tau \sqrt{3} S}{C_\tau R_{eH\_P}}\right)^2 = 1.0$$

with the corresponding buckling utilization factor defined as

$$\eta_{curved\_plate} = \frac{1}{\gamma_c}$$

where:

σ_{ax} Applied axial stress to the cylinder corresponding to the curved plate panel, in N/mm^2. In case of tensile axial stresses, $\sigma_{ax} = 0$.

σ_{tg} Applied tangential stress to the cylinder corresponding to the curved plate panel, in N/mm^2. In case of tensile tangential stresses, $\sigma_{tg} = 0$.

C_{ax}, C_{tg}, C_τ Buckling reduction factor of the curved plate panel, as defined in Table 4.

The stress multiplier factor, γ_c, of the curved plate panel need not be taken less than the stress multiplier factor, γ_c, for the expanded plane panel according to [2.2.1].

S35
(cont)

Figure 1: Transverse stiffened bilge plating

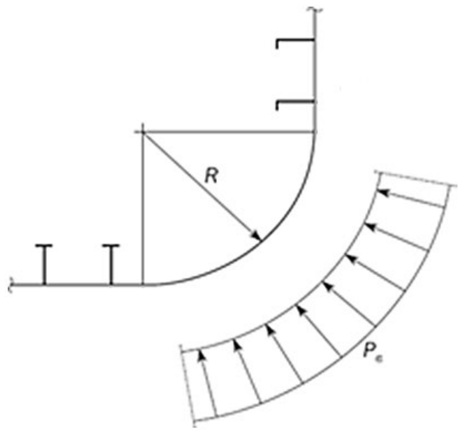
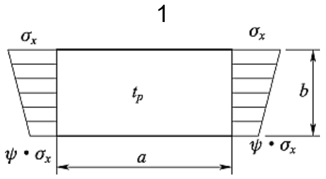
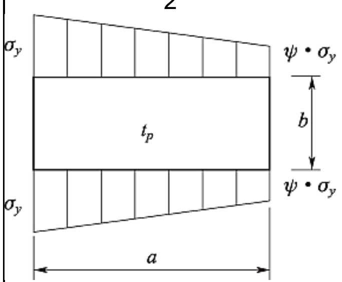


Table 3: Buckling factor and reduction factor for plane plate panels

<table>
<tr><th>Case</th><th>Stress ratio ψ</th><th>Aspect ratio α</th><th>Buckling factor K</th><th>Reduction factor C</th></tr>
<tr><td rowspan="3">1
</td><td>$1 \geq \psi \geq 0$</td><td colspan="2">$K_x = F_{long}\dfrac{8.4}{\psi + 1.1}$</td><td rowspan="3">When $\sigma_x \leq 0$, $C_x = 1$

When $\sigma_x > 0$,
$C_x = 1$ for $\lambda \leq \lambda_c$
$C_x = c\left(\frac{1}{\lambda} - \frac{0.22}{\lambda^2}\right)$ for $\lambda > \lambda_c$

where:
$c = (1.25 - 0.12\psi) \leq 1.25$
$\lambda_c = \dfrac{c}{2}\left(1 + \sqrt{1 - \dfrac{0.88}{c}}\right)$</td></tr>
<tr><td>$0 > \psi > -1$</td><td colspan="2">$K_x = F_{long}[7.63 - \psi(6.26 - 10\,\psi)]$</td></tr>
<tr><td>$\psi \leq -1$</td><td colspan="2">$K_x = F_{long}[5.975(1 - \psi)^2]$</td></tr>
<tr><td rowspan="5">2
</td><td rowspan="3">$1 \geq \psi \geq 0$</td><td colspan="2">$K_y = \dfrac{F_{tran} \cdot 2\left(1 + \frac{1}{\alpha^2}\right)^2}{1 + \psi + \frac{(1-\psi)}{100}\left(\frac{2.4}{\alpha^2} + 6.9 f_1\right)}$</td><td rowspan="5">When $\sigma_y \leq 0$, $C_y = 1$

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 $\lambda \geq \lambda c$

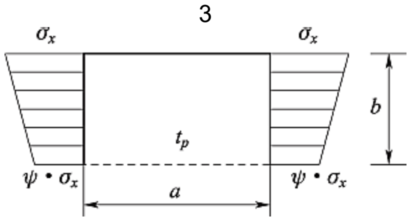
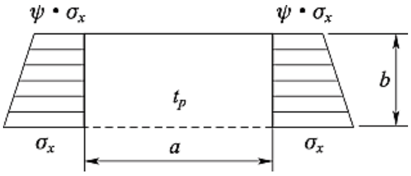
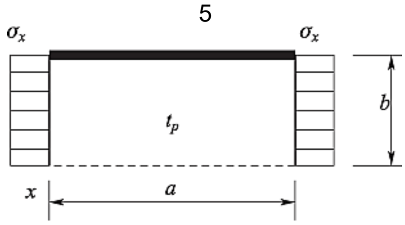
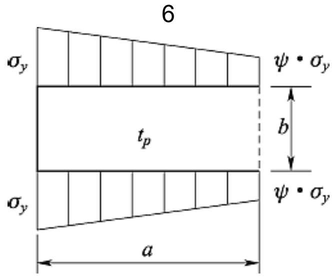
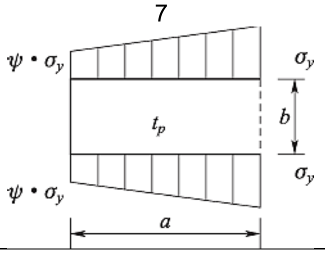
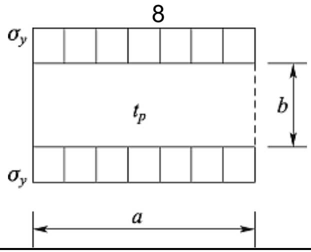
$\lambda_c = 0.5c\left(1 + \sqrt{1 - \frac{0.88}{c}}\right)$</td></tr>
<tr><td>$\alpha \leq 6$</td><td>$f_1 = (1 - \psi)(\alpha - 1)$</td></tr>
<tr><td>$\alpha > 6$</td><td>$f_1 = 0.6\left(1 - \frac{6\psi}{\alpha}\right)\left(\alpha + \frac{14}{\alpha}\right)$ but not greater than $14.5 - \frac{0.35}{\alpha^2}$</td></tr>
<tr><td rowspan="2">$1 - \dfrac{4a}{3} \leq \psi < 0$</td><td colspan="2">$K_y =$
$\dfrac{200 F_{tran}(1 + \beta^2)^2}{(1 - f_3)(100 + 2.4\beta^2 + 6.9 f_1 + 23 f_2)}$</td></tr>
<tr><td>$\alpha > 6(1 - \psi)$</td><td>$f_1 = 0.6\left(\dfrac{1}{\beta} + 14\beta\right)$
but not greater than
$14.5 - 0.35\beta^2$

$f_2 = f_3 = 0$</td></tr>
</table>

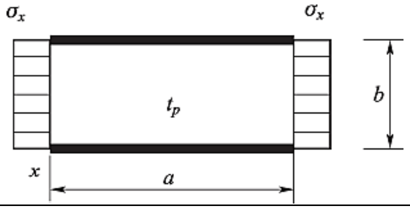
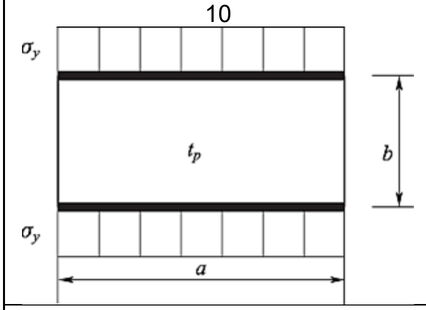
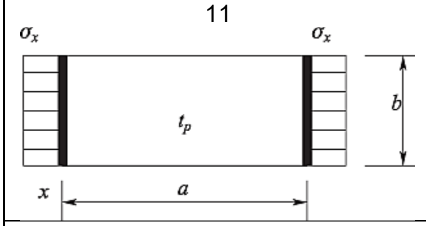
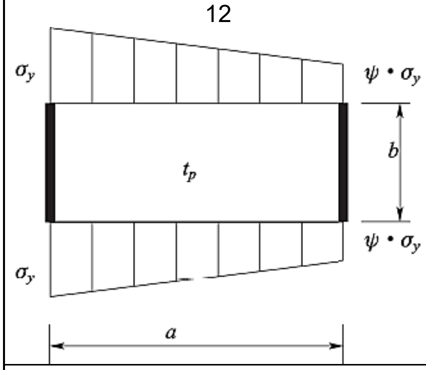
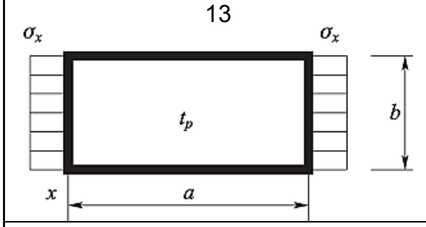
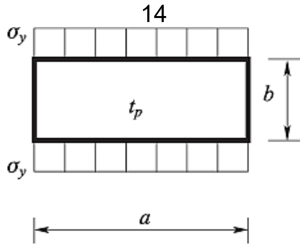
S35
(cont)

		$3(1-\Psi) \leq \alpha \leq 6(1-\Psi)$	$f_1 = \frac{1}{\beta} - 1$ $f_2 = f_3 = 0$	$F = \left(1 - \frac{\left(\frac{K}{0.91} - 1\right)}{\lambda_p^2}\right) c_1 \geq 0$ $\lambda_p^2 = \lambda^2 - 0.5$ for $1 \leq \lambda_p^2 \leq 3$ c_1 as defined in [2.2.3] $H = \lambda - \frac{2\lambda}{c\left(T + \sqrt{T^2 - 4}\right)} \geq R$ $T = \lambda + \frac{14}{15\lambda} + \frac{1}{3}$
		$1.5(1-\Psi) \leq \alpha < 3(1-\Psi)$	$f_1 = \frac{1}{\beta} - (2 - \omega\beta)^4$ $-9(\omega\beta - 1)(\frac{2}{3} - \beta)$ $f_2 = f_3 = 0$	
		$1 - \Psi \leq \alpha < 1.5(1-\Psi)$	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 $\alpha \leq 1.5$ $f_1 = 2\left(\frac{1.5}{1-\psi} - 1\right)\left(\frac{1}{\beta} - 1\right)$ $f_2 = \frac{\psi(1 - 16f_4^2)}{1 - \alpha}$ $f_3 = 0$ $f_4 = (1.5 - Min(1.5, \alpha))^2$	
		$0.75(1-\Psi) \leq \alpha < 1 - \Psi$	$f_1 = 0$ $f_2 = 1 + 2.31(\beta - 1)$ $-48\left(\frac{4}{3} - \beta\right) f_4^2$ $f_3 = 3f_4(\beta - 1) - \left(\frac{f_4}{1.81} - \frac{\alpha - 1}{1.31}\right)$ $f_4 = (1.5 - Min(1.5, \alpha))^2$	
	$\psi < 1 - \frac{4\alpha}{3}$	$K_y = 5.972 F_{tran} \frac{\beta^2}{1 - f_3}$ where $f_3 = f_5\left(\frac{f_5}{1.81} + \frac{1 + 3\psi}{5.24}\right)$ $f_5 = \frac{9}{16}(1 + Max(-1, \psi))^2$		

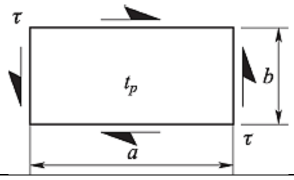
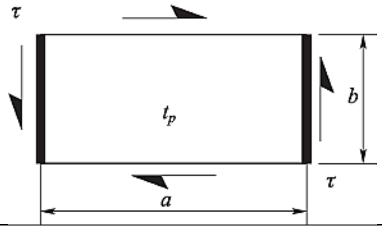
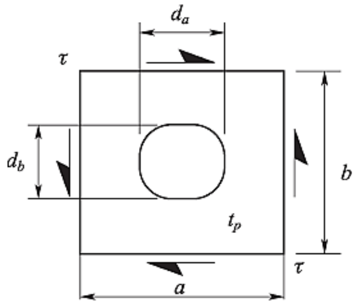
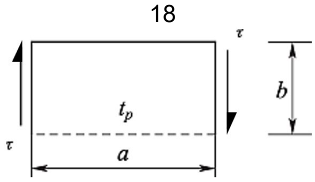
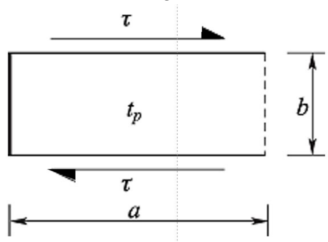
S35
(cont)

Case	Stress ratio ψ	Aspect ratio α	Buckling factor *K*	Reduction factor *C*
3 	$1 \ge \psi \ge 0$		$K_x = \frac{4(0.425 + 1/\alpha^2)}{3\psi + 1}$	For UP-A: $C_x = 1$ for $\lambda \le 0.75$ $C_x = \frac{0.75}{\lambda}$ for $\lambda > 0.75$ For UP-B: $C_x = 1$ for $\lambda \le 0.7$ $C_x = \frac{1}{\lambda^2 + 0.51}$ for $\lambda > 0.7$
	$0 > \psi > -1$		$K_x = 4\left(0.425 + \frac{1}{\alpha^2}\right)(1 + \psi) - 5\psi(1 - 3.42\psi)$	
4 	$1 \ge \psi \ge -1$		$K_x = \left(0.425 + \frac{1}{\alpha^2}\right)\frac{3 - \psi}{2}$	
5 	—	$\alpha \ge 1.64$	$K_x = 1.28$	
		$\alpha < 1.64$	$K_x = \frac{1}{\alpha^2} + 0.56 + 0.13\alpha^2$	
6 	$1 \ge \psi \ge 0$		$K_y = \frac{4(0.425 + \alpha^2)}{(3\psi + 1)\alpha^2}$	For UP-A: $C_y = 1$ for $\lambda \le 0.75$ $C_y = \frac{0.75}{\lambda}$ for $\lambda > 0.75$ For UP-B: $C_y = 1$ for $\lambda \le 0.7$ $C_y = \frac{1}{\lambda^2 + 0.51}$ for $\lambda > 0.7$
	$0 > \psi \ge -1$		$K_y = 4(0.425 + \alpha^2)(1 + \psi)\frac{1}{\alpha^2} - 5\psi(1 - 3.42\psi)\frac{1}{\alpha^2}$	
7 	$1 \ge \psi \ge -1$		$K_y = (0.425 + \alpha^2)\frac{(3 - \psi)}{2\alpha^2}$	
8 	—		$K_y = 1 + \frac{0.56}{\alpha^2} + \frac{0.13}{\alpha^4}$	

S35
(cont)

Case	Stress ratio ψ	Aspect ratio α	Buckling factor K	Reduction factor C
9 	—		$K_x = 6.97$	$C_x = 1$ for $\lambda \le 0.83$ $C_x = 1.13\left(\frac{1}{\lambda} - \frac{0.22}{\lambda^2}\right)$ for $\lambda > 0.83$
10 	—		$K_y = 4 + \dfrac{2.07}{\alpha^2} + \dfrac{0.67}{\alpha^4}$	$C_y = 1$ for $\lambda \le 0.83$ $C_y = 1.13\left(\frac{1}{\lambda} - \frac{0.22}{\lambda^2}\right)$ for $\lambda > 0.83$
11 	—	$\alpha \ge 4$	$K_x = 4$	$C_x = 1$ for $\lambda \le 0.83$ $C_x = 1.13\left(\frac{1}{\lambda} - \frac{0.22}{\lambda^2}\right)$ for $\lambda > 0.83$
		$\alpha < 4$	$K_x = 4 + 2.74\left(\dfrac{4-\alpha}{3}\right)^4$	
12 	—		$K_y = K_y$ determined as per case 2	For $\alpha < 2$ $C_y = C_{y2}$ For $\alpha \ge 2$ $C_y = \left(1.06 + \dfrac{1}{10\alpha}\right) C_{y2}$ where: C_{y2} : C_y determined as per case 2
13 	—	$\alpha \ge 4$	$K_x = 6.97$	$C_x = 1$ for $\lambda \le 0.83$ $C_x = 1.13\left(\frac{1}{\lambda} - \frac{0.22}{\lambda^2}\right)$ for $\lambda > 0.83$
		$\alpha < 4$	$K_x = 6.97 + 3.1\left(\dfrac{4-\alpha}{3}\right)^4$	
14 	—		$K_y = \dfrac{6.97}{\alpha^2} + \dfrac{3.1}{\alpha^2}\left(\dfrac{4 - 1/\alpha}{3}\right)^4$	$C_y = 1$ for $\lambda \le 0.83$ $C_y = 1.13\left(\frac{1}{\lambda} - \frac{0.22}{\lambda^2}\right)$ for $\lambda > 0.83$

S35
(cont)

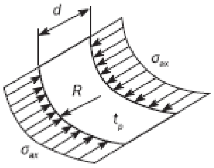
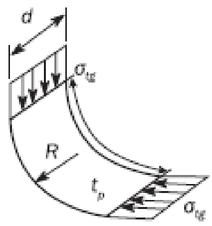
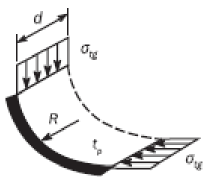
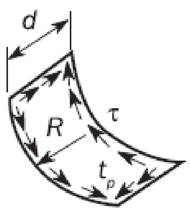
Case	Stress ratio ψ	Aspect ratio α	Buckling factor K	Reduction factor C
15 	–		$K_\tau = \sqrt{3}\left(5.34 + \frac{4}{\alpha^2}\right)$	$C_\tau = 1$ for $\lambda \le 0.84$ $C_\tau = \frac{0.84}{\lambda}$ for $\lambda > 0.84$
16 	–		$K_\tau = \sqrt{3}\left[5.34 + Max\left(\frac{4}{\alpha^2}, \frac{7.15}{\alpha^{2.5}}\right)\right]$	
17 	–		$K_\tau = K_{\tau case15} r$ $K_{\tau case15}$: K_τ according to case 15 r : Opening reduction factor taken as $r = \left(1 - \frac{d_a}{a}\right)\left(1 - \frac{d_b}{b}\right)$ with $\frac{d_a}{a} \le 0.7$ *and* $\frac{d_b}{b} \le 0.7$	
18 	–		$K_\tau = \sqrt{3}\left(0.6 + \frac{4}{\alpha^2}\right)$	$C_\tau = 1$ for $\lambda \le 0.84$ $C_\tau = \frac{0.84}{\lambda}$ for $\lambda > 0.84$
19 	–		$K_\tau = 8$	

Edge boundary conditions:
Plate edge free
Plate edge simply supported
Plate edge clamped

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

S35
(cont)

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

Case	Aspect ratio	Buckling factor *K*	Reduction factor *C*
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: $C_{ax} = 1$ for $\lambda \le 0.25$; $C_{ax} = 1.233 - 0.933\lambda$ for $0.25 < \lambda \le 1$; $C_{ax} = 0.3/\lambda^3$ for $1 < \lambda \le 1.5$; $C_{ax} = 0.2/\lambda^2$ for $\lambda > 1.5$
	$\frac{d}{R} > 0.5\sqrt{\frac{R}{t_p}}$	$K = 0.267\frac{d^2}{Rt_p}\left[3 - \frac{d}{R}\sqrt{\frac{t_p}{R}}\right] \ge 0.4\frac{d^2}{Rt_p}$	For curved single fields, e.g. bilge strake, which are bounded by plane panels as shown in Figure 1: $C_{ax} = \frac{0.65}{\lambda^2} \le 1.0$
2 	$\frac{d}{R} \le 1.63\sqrt{\frac{R}{t_p}}$	$K = \frac{d}{\sqrt{Rt_p}} + 3\frac{(Rt_p)^{0.175}}{d^{0.35}}$	For general application: $C_{tg} = 1$ 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 $\lambda > 1.2$
	$\frac{d}{R} > 1.63\sqrt{\frac{R}{t_p}}$	$K = 0.3\frac{d^2}{R^2} + 2.25\left(\frac{R^2}{dt_p}\right)^2$	For curved single fields, e.g. bilge strake, which are bounded by plane panels as shown in Figure 1: $C_{tg} = \frac{0.8}{\lambda^2} \le 1.0$
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 load case 2.
	$\frac{d}{R} > \sqrt{\frac{R}{t_p}}$	$K = 0.3\frac{d^2}{R^2} + 0.291\left(\frac{R^2}{dt_p}\right)^2$	
4 	$\frac{d}{R} \le 8.7\sqrt{\frac{R}{t_p}}$	$K = \sqrt{3}\sqrt{28.3 + \frac{0.67d^3}{R^{1.5}t_p^{1.5}}}$	$C_\tau = 1$ 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 $\lambda > 1.2$
	$\frac{d}{R} > 8.7\sqrt{\frac{R}{t_p}}$	$K = \sqrt{3}\frac{0.28d^2}{R\sqrt{Rt_p}}$	

Explanations for boundary conditions:

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

———————— Plate edge simply supported.

▬▬▬▬▬▬ Plate edge clamped.

S35 (cont)

2.2.7 Applied normal and shear stresses to plate panels

The normal stress, σ_x and σ_y, in N/mm^2, to be applied for the overall stiffened panel capacity and the plate panel capacity calculations as given in [2.1.1] and [2.2.1] respectively, are to be taken as follows:

- For FE analysis, the reference stresses as defined in Sec.4 [2.4].

- For prescriptive assessment of the overall stiffened panel capacity and the plate panel capacity, the axial or transverse compressive stresses calculated according to the Relevant UR-S, at load calculation points of the considered stiffener or the considered elementary plate panel, as defined in item a) and item b) of Sec 3, [1.2.1] respectively. However, in case of transverse stiffening arrangement, the transverse compressive stress used for the assessment of the overall stiffened panel capacity is to be taken as the compressive stress calculated at load calculation points of the stiffener attached plating, as defined in item a) of Sec 3, [1.2.1].

- For grillage analysis where the stresses are obtained based on beam theory, the stresses taken as:

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

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

Where

σ_{xb}, σ_{yb} Stress, in N/mm^2, from grillage beam analysis respectively along x or y axis of the plate attached to the PSM web.

The shear stress τ, in N/mm^2, to be applied for the overall stiffened panel capacity and the plate panel capacity calculations as given in [2.1.1] and [2.2.1] respectively, are to be taken as follows:

- For FE analysis, the reference shear stresses as defined in Sec 4, [2.4].

- For prescriptive assessment of the plate panel capacity, the shear stresses calculated according to the Relevant UR-S, at load calculation points of the considered elementary plate panel, as defined in item a) of Sec 3, [1.2.1].

- For prescriptive assessment of the overall stiffened panel capacity, the shear stresses calculated according to the Relevant UR-S, at the following load calculation point:
  - 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, $\tau = 0$ in the plate attached to the PSM web.

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*).

S35
(cont)

2.3.2 Web thickness of flat bar

For accounting the decrease of the stiffness due to local lateral deformation, the effective web thickness of flat bar stiffener, in mm, is to be used in [2.1] and [2.3.4] for the calculation of the net sectional area, A_s, the net section modulus, Z, and the moment of inertia, I, of the stiffener and is taken as:

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

2.3.3 Idealisation of bulb profile

Bulb profiles are to be considered as equivalent angle profiles. The net dimensions of the equivalent built-up section are to be obtained, in mm, from the following formulae.

$$h_w = h'_w - \frac{h'_w}{9.2} + 2$$

$$b_f = \alpha \left(t'_w + \frac{h'_w}{6.7} - 2 \right)$$

$$t_f = \frac{h'_w}{9.2} - 2$$

$$t_w = t'_w$$

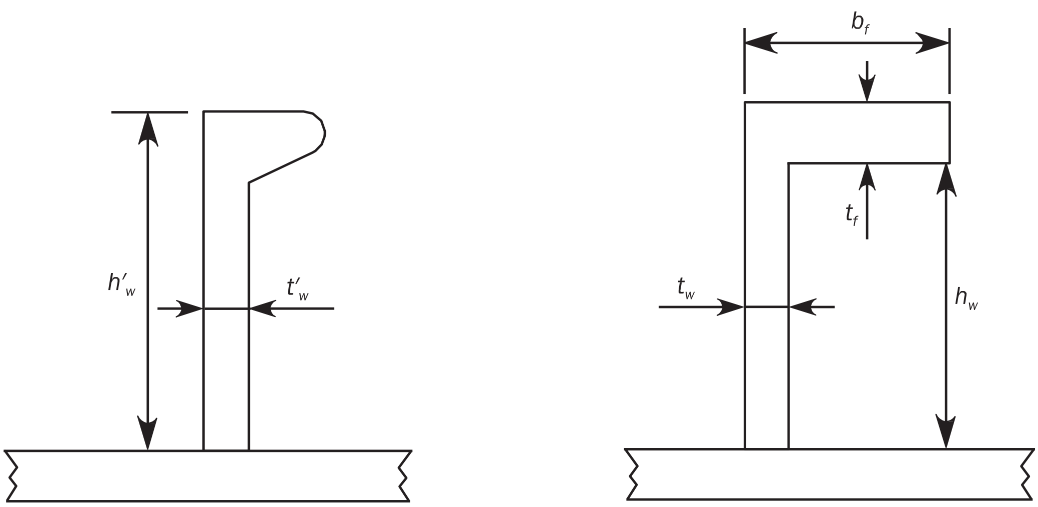
where:

h'_w, t'_w Net height and thickness of a bulb section, in mm, as shown in Figure 2.

α Coefficient equal to $\alpha = 1.1 + \frac{(120 - h'_w)^2}{3000}$ for $h'_w \le 120$

$\alpha = 1.0$ for $h'_w > 120$

Figure 2: Idealisation of bulb stiffener



Bulb flat   Equivalent angle bar

S35 (cont)

2.3.4 Ultimate buckling capacity

When $\sigma_a + \sigma_b + \sigma_w > 0$ while initially setting $\gamma = 1$, the ultimate buckling capacity for stiffeners is to be checked according to the following interaction formula:

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

with the corresponding buckling utilization factor defined as

$$\eta_{stiffener} = \frac{1}{\gamma_c}$$

where:

σ_a Effective axial stress, in N/mm^2, 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/mm^2, 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 calculated according to Sec 3, [2.2.1] at load calculation point of the stiffener, as defined in Sec 3, [1.2.1].
- 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/mm^2

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

σ_b Bending stress in the stiffener, in N/mm^2:

$$\sigma_b = \frac{M_0 + M_1 + M_2}{1000 Z}$$

Z Net section modulus of stiffener, in cm^3, including effective width of plating according to [2.3.5], to be taken as:

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

M_2 Bending moment, in Nmm, due to eccentricity of sniped stiffeners, to be taken as

- $M_2 = 0$ for continuous stiffeners
- $M_2 = C_{snip} w_{na} \gamma \sigma_x (A_p + A_s)$ for stiffeners sniped at one or both ends.

C_{snip} Coefficient to account for the end effect of the stiffener sniped at one or both ends, to be taken as

- $C_{snip} = -1.2$ for stiffener induced failure (*SI*)
- $C_{snip} = 1.2$ for plate induced failure (*PI*).

S35 (cont)

M_1 Bending moment, in Nmm, due to the lateral load *P*:

- $M_1 = C_i \frac{|P|sl^2}{24\times10^3}$ for continuous stiffener
- $M_1 = C_i \frac{|P|sl^2}{8\times10^3}$ for sniped stiffener
- $M_1 = C_i \frac{|P|sl^2}{14.2\times10^3}$ for stiffener sniped at one end and continuous at the other end

P Lateral load, in kN/m^2.

- For FE analysis, P is the average pressure 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 Sec 3, [1.2.1].

C_i Pressure coefficient:

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

C_{PI} 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_0 Bending moment, in Nmm, due to the lateral deformation *w* of stiffener:

$$M_0 = F_E C_{sl} \frac{\gamma}{\gamma_{GEB} - \gamma} w_0 \text{ with precondition } \gamma_{GEB} - \gamma > 0$$

γ_{GEB} Stress multiplier factor of global elastic buckling capacity as defined in [2.1].

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

$$F_E = \left(\frac{\pi}{l}\right)^2 EI \cdot 10^4$$

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

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

t_p Net thickness of plate, 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.

C_{sl} Deformation reduction factor to account for global slenderness, to be taken as:

- $C_{sl} = 1 - \frac{1}{12}\lambda_G^4$ for $\lambda_G \leq 1.56$
- $C_{sl} = 3 / \lambda_G^4$ for $\lambda_G > 1.56$

S35 (cont)

λ_G The reference degree of global slenderness of the stiffened panel, to be taken as
$\lambda_G = \sqrt{\frac{\gamma_{ReH}}{\gamma_{GEB}}}$ with $\gamma_{ReH} = \frac{\text{Min}\,(R_{eH\_P},\ R_{eH\_S})}{\sqrt{\sigma_{x,av}^2+\sigma_y^2-\sigma_{x,av}\sigma_y+3\tau^2}}$

$\sigma_{x,av}$ Average stress for both plate and stiffener as defined in [2.1.2].

σ_y Applied transverse stress to the plate panel as defined in [2.1.1].

τ Applied shear stress to the plate panel as defined in [2.1.1].

w_0 Assumed imperfection, in mm, to be taken as:
$w_0 = l/1000.$

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

- For stiffener induced failure (*SI*)
  - For $\sigma_a > 0$

    $$\sigma_w = E y_w e_f \Phi_0 \left(\frac{m_{tor}\pi}{l_{tor}}\right)^2 \left(\frac{1}{1-\frac{\gamma\sigma_a}{\sigma_{ET}}}-1\right) \text{ with precondition } \sigma_{ET}-\gamma\sigma_a > 0$$

  - For $\sigma_a \le 0$

    $\sigma_w = 0$

- For plate induced failure (*PI*)

  $\sigma_w = 0$

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

- $y_w = \frac{t_w}{2}$ for flat bar
- $y_w = b_f - \frac{h_w t_w^2 + t_f b_f^2}{2A_s}$ for angle and bulb profiles
- $y_w = b_{f-out} + 0.5t_w - \frac{h_w t_w^2 + t_f\left(b_f^2 - 2b_f d_f\right)}{2A_s}$ for L2 profile
- $y_w = \frac{b_f}{2}$ for T profile.

Φ_0 Coefficient taken as:

$$\Phi_0 = \frac{l_{tor}}{m_{tor} h_w} 10^{-4}$$

σ_{ET} : Reference stress for torsional buckling, in N/mm², to be taken as:

$$\sigma_{ET} = \frac{E}{I_p}\left[\left(\frac{m_{tor}\pi}{l_{tor}}\right)^2 I_\omega \cdot 10^2 + \frac{1}{2(1+\nu)} I_T + \left(\frac{l_{tor}}{m_{tor}\pi}\right)^2 \varepsilon \cdot 10^{-4}\right]$$

I_p Net polar moment of inertia of the stiffener, in cm⁴, about point *C* as shown in Sec 1, Figure 3, as defined in Table 5.

I_T Net St. Venant's moment of inertia of the stiffener, in cm⁴, as defined in Table 5.

S35 (cont)

I_ω Net sectorial moment of inertia of the stiffener, in cm^6, about point C as shown in Sec 1, Figure 3, as defined in Table 5.

l_{tor} Stiffener span, distance equal to spacing between primary supporting members, i.e. $l_{tor} = l$. When the stiffener is supported by tripping brackets, l_{tor} should be taken as the maximum spacing between the adjacent primary supporting members and fitted tripping brackets.

m_{tor} Number of half waves, taken as a positive integer so as to give smallest reference stress for torsional buckling.

ε Degree of fixation, in mm^2, to be taken as:

- $\varepsilon = \left(\frac{3b}{t_p^3} + \frac{2h_w}{t_w^3}\right)^{-1}$ for bulb, angle, L2 and T profiles;
- $\varepsilon = \left(\frac{t_p^3}{3\mathrm{b}}\right)$ for flat bars.

A_w : Net web area, in mm^2.

A_f : Net flange area, in mm^2.

Table 5: Moments of inertia

	Flat bars[(1)]	Bulb, angle, L2 and T profiles
I_p	$\frac{h_w^3 t_w}{3 \cdot 10^4}$	$\left(\frac{A_w(e_f - 0.5t_f)^2}{3} + A_f e_f^2\right) 10^{-4}$
I_T	$\frac{h_w t_w^3}{3 \cdot 10^4}\left(1 - 0.63\frac{t_w}{h_w}\right)$	$\frac{(e_f - 0.5t_f)t_w^3}{3 \cdot 10^4}\left(1 - 0.63\frac{t_w}{e_f - 0.5t_f}\right) + \frac{b_f t_f^3}{3 \cdot 10^4}\left(1 - 0.63\frac{t_f}{b_f}\right)$
I_ω	$\frac{h_w^3 t_w^3}{36 \cdot 10^6}$	For bulb, angle and L2 profiles[(2)]: $\frac{A_f^3 + A_w^3}{36 \cdot 10^6} + \frac{e_f^2}{10^6}\left(\frac{A_f b_f^2 + A_w t_w^2}{3} - \frac{\left(A_f(b_f - 2d_f) + A_w t_w\right)^2}{4(A_f + A_w)} - A_f d_f(b_f - d_f)\right)$ For T profile: $\frac{b_f^3 t_f e_f^2}{12 \cdot 10^6}$

(1) t_w is the net web thickness, in mm. t_{w_red} as defined in [2.3.2] is not to be used in this table.

(2) d_f is to be taken as 0 for bulb and angle profiles.

S35
(cont)

2.3.5 Effective width of attached plating

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

- For $\sigma_x > 0$:
  - For FE analysis,

    $b_{eff} = Min(C_x b, \chi_s s)$
  - For prescriptive assessment,

    $$b_{eff} = Min\left(\frac{C_{x1} b_1 + C_{x2} b_2}{2}, \chi_s s\right)$$
- For $\sigma_x \le 0$:

  $b_{eff} = \chi_s s$

where:

χ_s : Effective width coefficient to be taken as:

$$\chi_s = \frac{1.12}{1 + \frac{1.75}{\left(\frac{\ell_{eff}}{s}\right)^{1.6}}} \le 1.0 \quad \text{for} \quad \frac{\ell_{eff}}{s} \ge 1$$

$$\chi_s = 0.407 \frac{\ell_{eff}}{s} \quad \text{for} \quad \frac{\ell_{eff}}{s} < 1$$

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

$$\ell_{eff} = \frac{l}{\sqrt{3}} \qquad \text{for stiffener fixed at both ends.}$$

$$\ell_{eff} = 0.75l \quad \text{for stiffener simply supported at one end and fixed at the other.}$$

$$\ell_{eff} = l \qquad \text{for stiffener simply supported at both ends.}$$

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, σ_x is to be corrected according to the following formulae:

- If $\sigma_x < \nu\sigma_y$
  $\sigma_{xcor} = 0$

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

S35
(cont)

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 Table 6.

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 Table 6.

For the application of Table 6, the weighted average shear stress is to be taken as:

- Opening modelled in primary supporting members:
  τ_{av} Weighted average shear stress, in N/mm^2, in the area of web plate being considered, i.e. *P1*, *P2*, or *P3* as shown in Table 6.

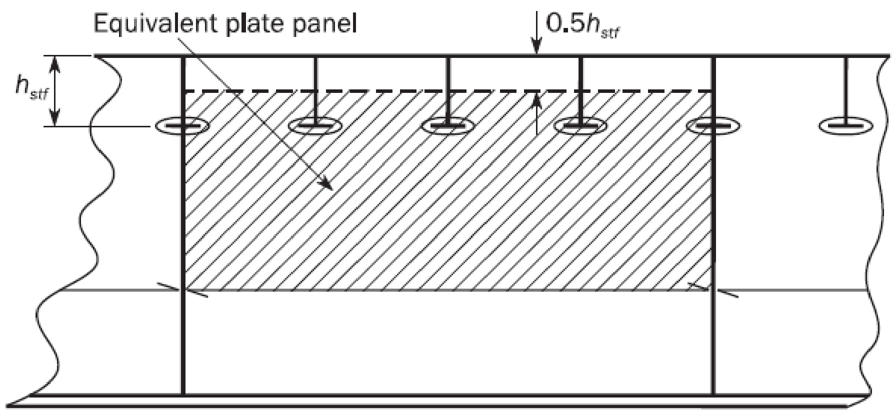
- Opening not modelled in primary supporting members:
  τ_{av} Weighted average shear stress, in N/mm^2, given in Table 6.

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 Table 6.

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 Figure 3.

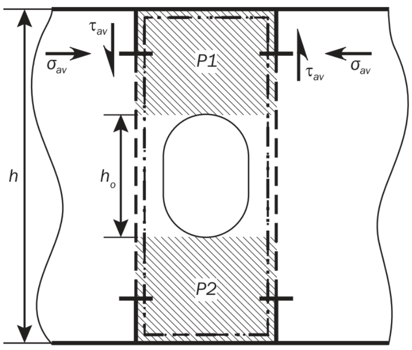
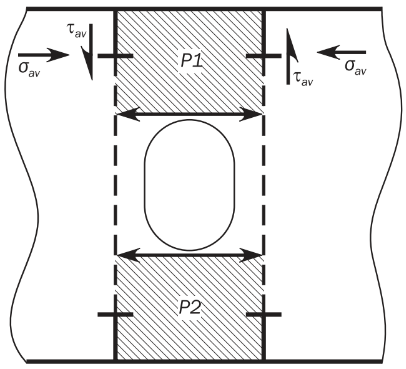
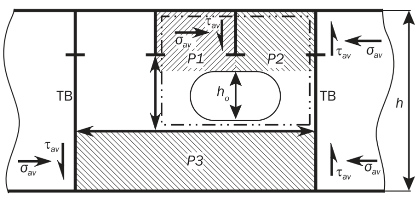
Figure 3: Web plate idealization



S35 (cont)

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

Table 6: Reduction factors

Configuration[1]	C_x, C_y	C_τ	
		Opening modelled in PSM	**Opening not modelled in PSM**
(a) Without edge reinforcements:[2] 	Separate reduction factors are to be applied to areas *P1* and *P2* using case 3 or case 6 in Table 3, with edge stress ratio: $\psi = 1.0$	Separate reduction factors are to be applied to areas *P1* and *P2* using case 18 or case 19 in Table 3.	When case 17 of Table 3 is applicable: A common reduction factor is to be applied to areas *P1* and *P2* using case 17 in Table 3 with: $\tau_{av} = \tau_{av}\,(web)$ When case 17 of Table 3 is not applicable: Separate reduction factors are to be applied to areas *P1* and *P2* using case 18 or case 19 in Table 3 with: $\tau_{av} = \tau_{av}\,(web)h/\,(h - h_0)$
(b) With edge reinforcements: 	Separate reduction factors are to be applied for areas *P1* and *P2* using C_X for case 1 or C_Y for case 2 in Table 3 with stress ratio: $\psi = 1.0$	Separate reduction factors are to be applied for areas *P1* and *P2* using case 15 in Table 3.	Separate reduction factors are to be applied to areas *P1* and *P2* using case 15 in Table 3 with: $\tau_{av} = \tau_{av}\,(web)h/\,(h - h_0)$
(c) Example of hole in web: 	Panels *P1* and *P2* are to be evaluated in accordance with (a). Panel *P3* is to be evaluated in accordance with (b).		
Where: *h* Height, in m, of the web of the primary supporting member in way of the opening.			

S35
(cont)

h_0	Height in m, of the opening measured in the depth of the web.
$\tau_{av}\ (web)$	Weighted average shear stress, in N/mm^2, over the web height h of the primary supporting member.
Note (1)	Web panels to be considered for buckling in way of openings are shown shaded and numbered P1, P2, etc.
Note (2)	For a PSM web panel with opening and without edge reinforcements as shown in configuration (a), the applicable buckling assessment method depends on its specific boundary conditions. If one of the long edges along the face plate or along the attached plating is not subject to "inline support", i.e. the edge is free to pull in, Method B should be applied. In other cases, typically such as when the short plate edge is attached to the plate flanges, Method A is applicable.

2.5 Stiffened Panels with U-type Stiffeners

2.5.1 Local plate buckling

For stiffened panels with U-type stiffeners, local plate buckling is to be checked for each of the plate panels EPP b_1, b_2, b_f and h_w (see Sec 1, Figure 4) separately as follows:

- The attached plate panels EPP b_1 and b_2 are to be assessed using SP-A model, where in the calculation of buckling factors K_x as defined in Case 1 of Table 3, the correction factor F_{long} for U-type stiffeners as defined in Table 2 is to be used; and in the calculation of K_y as defined in Case 2 of Table 3, the F_{tran} for U-type stiffeners as defined in [2.2.5] is to be used.

- The face plate and web plate panels b_f and h_w are to be assessed using UP-B model with $F_{long} = 1$ and $F_{tran} = 1$.

2.5.2 Overall stiffened panel buckling and stiffener buckling

For a stiffened panel with U-type stiffeners, the overall buckling capacity and ultimate capacity of the stiffeners are to be checked with warping stress $\sigma_w = 0$, and with bending moment of inertia including effective width of attached plating being calculated based on the following assumptions:

- The two web panels of a U-type stiffener are to be taken as perpendicular to the attached plate with thickness equal to t_w and height equal to the distance between the attached plate and the face plate of the stiffener.

- Effective width of the attached plating, b_{eff}, taken as the sum of the b_{eff} calculated for the EPP b_1 and b_2 respectively according to SP-A model.

- Effective width of the attached plating of a stiffener without shear lag effect, b_{eff1}, taken as the sum of the b_{eff1} calculated for the EPP b_1 and b_2 respectively.

S35 (cont)

3. Buckling capacity of column structures

3.1 Column Buckling of Corrugations

3.1.1 Buckling utilisation factor

The column buckling utilisation factor, η, for axially compressed corrugations is to be taken as:

$$\eta_{column} = \frac{\sigma_{av}}{\sigma_{cr}}$$

where:

σ_{av} Average axial compressive stress in the member, in N/mm^2.

σ_{cr} Minimum critical buckling stress, in N/mm^2, taken as:

- $\sigma_{cr} = \sigma_E \quad \text{for} \quad \sigma_E \leq 0.5 R_{eH\_S}$
- $\sigma_{cr} = \left(1 - \frac{R_{eH\_S}}{4\sigma_E}\right) R_{eH\_S} \quad \text{for} \quad \sigma_E > 0.5 R_{eH\_S}$

σ_E Elastic column compressive buckling stress, in N/mm^2, according to [3.1.2].

R_{eH_S} Specified minimum yield stress of the considered member, in N/mm^2. 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_E in N/mm^2 of members subject to axial compression is to be taken as:

$$\sigma_E = \pi^2 E f_{end} \frac{I}{A l_{pill}^2} 10^{-4}$$

where:

I Net moment of inertia about the weakest axis of the cross section, in cm^4.

A Net cross-sectional area of the member, in cm^2.

l_{pill} Unsupported length of the member, in m.

f_{end} End constraint factor, corresponding to simply supported ends is to be applied except for fixed end support to be used in way of stool with width exceeding 2 times the depth of the corrugation, taken as:

- $f_{end} = 1.0$ 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.

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 section provides a method to determine stress distribution along edges of the considered buckling panel by second-order polynomial curve, by linear distribution using 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 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 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 of a curved panel are to be taken as defined in [2.2].

S35 (cont)

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 second order polynomial curve as:

  $\sigma_x = Cx^2 + Dx + E$

  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.

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

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

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

  The unknown coefficients *C*, *D* and *E* can be obtained by solving the 3 above equations.

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

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

  If $-\frac{D}{2C} < \frac{b}{2}$ or $-\frac{D}{2C} > a - \frac{b}{2}$, σ_{x3} is to be ignored. Otherwise, σ_{x3} is taken as:

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

  where:

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

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

S35 (cont)

The longitudinal stress is to be taken as:
$\sigma_x = Max(\sigma_{x1}, \sigma_{x2}, \sigma_{x3})$

The edge stress ratio is to be taken as:
$\psi_x = 1$

- For overall stiffened panel buckling and stiffener buckling assessments, the longitudinal stress σ_x applied on the shorter edge of the attached plate is to be taken as:

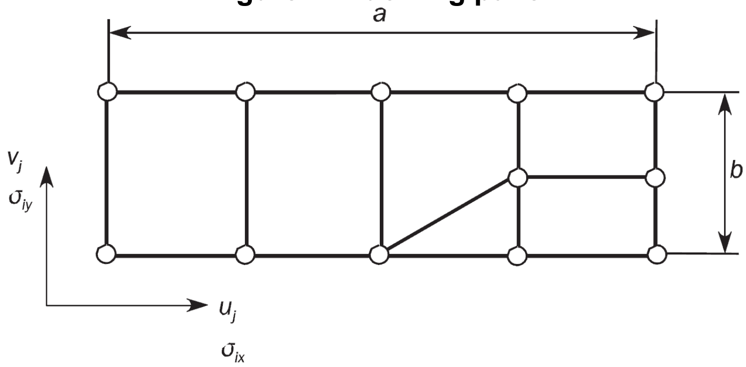
$$\sigma_x = \frac{\sum_{i=1}^{n} A_i \sigma_{xi}}{\sum_{i=1}^{n} A_i}$$

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 elements up to the shorter edges of the considered buckling panel.

Figure 1: Buckling panel



The distribution of $\sigma_y(x)$ is assumed as straight line. Therefore: $\sigma_y(x) = A + Bx$

The best fitting curve $\sigma_y(x)$ is to be obtained by the least square method minimising the square error Π considering 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.

$$\frac{\partial\Pi}{\partial A} = 2\sum_{i=1}^{n} A_i [\sigma_{yi} - (A + Bx_i)] = 0$$

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

The unknown coefficients A and B are obtained by solving the 2 above equations and are given as follow:

$$A = \frac{\left(\sum_{i=1}^{n} A_i \sigma_{yi}\right)\left(\sum_{i=1}^{n} A_i x_i^2\right) - \left(\sum_{i=1}^{n} A_i x_i\right)\left(\sum_{i=1}^{n} A_i x_i \sigma_{yi}\right)}{\left(\sum_{i=1}^{n} A_i\right)\left(\sum_{i=1}^{n} A_i x_i^2\right) - \left(\sum_{i=1}^{n} A_i x_i\right)^2}$$

S35 (cont)

$$B = \frac{\left(\sum_{i=1}^{n} A_i\right)\left(\sum_{i=1}^{n} A_i x_i \sigma_{yi}\right) - \left(\sum_{i=1}^{n} A_i x_i\right)\left(\sum_{i=1}^{n} A_i \sigma_{yi}\right)}{\left(\sum_{i=1}^{n} A_i\right)\left(\sum_{i=1}^{n} A_i x_i^2\right) - \left(\sum_{i=1}^{n} A_i x_i\right)^2}$$

The transverse stress is to be taken as:

$$\sigma_y = max(A, A + Ba)$$

The edge stress ratio is to be taken as:

$$\psi_y = \frac{min(A, A + Ba)}{max(A, A + Ba)} \text{ for } \sigma_y > 0$$

$$\psi_y = 1\ for\ \sigma_y \leq 0.$$

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

$$\psi_x = 1$$

$$\psi_y = 1$$

End of Document