Amendment on 27 June 2024
Resolved by Technical Committee on 30 January 2024

Buckling Strength Assessments of Ship Structural Elements

Object of Amendment

Rules for the Survey and Construction of Steel Ships Part C

Reason for Amendment

IACS Unified Requirements (UR) S11 and S11A specify requirements for hull girder buckling strength assessments, and UR S21 and S21A specify requirements for hatch cover buckling strength assessments. In addition, the IACS Common Structural Rules for Bulk Carriers and Oil Tankers (CSR-BC&OT) also specify the requirements for buckling strength assessments. ClassNK has already incorporated UR S11, S11A, S21 and S21A into Rules for the Survey and Construction of Steel Ships Part C and CSR-BC&OT into Rules for the Survey and Construction of Steel Ships Part CSR-B&T.

Since buckling strength assessment methods specified in each of the above URs and CSR-BC&OT were different, IACS decided to harmonise them based on the buckling assessment methods specified in the CSR-BC&OT to unify assessment methods and adopted UR S35 in February 2023 as result.

Accordingly, relevant requirements are amended in accordance with UR S35.

Outline of the Amendment

Specify general requirements related to buckling strength assessments in the Rules for the Survey and Construction of Steel Ships Part C Annex.

Effective Date and application

This amendment applies to ships for which the date of contract for construction is on or after 1 July 2024. This includes those ships to which Part C of the Rules for the Survey and Construction of Steel Ships applied prior to its comprehensive revision.

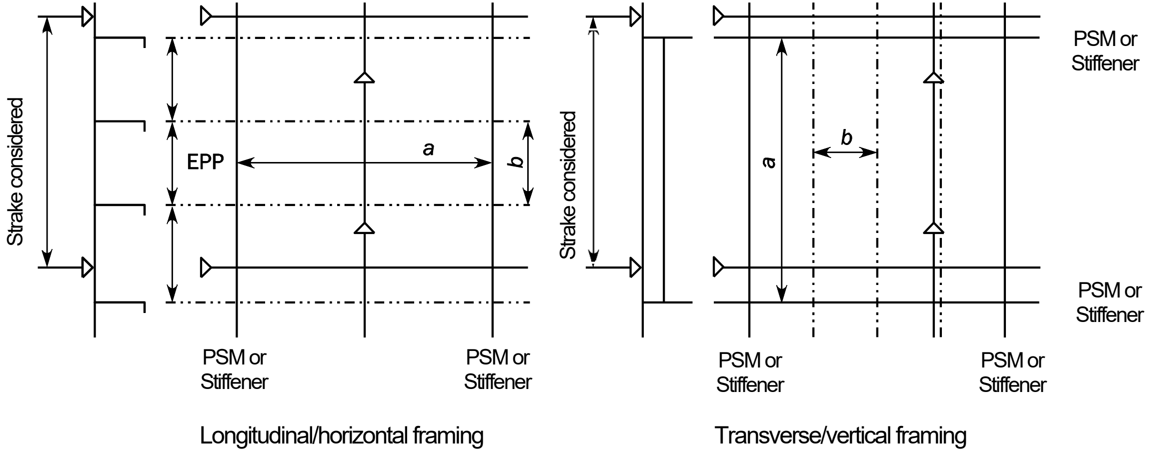
ID: DH23-06

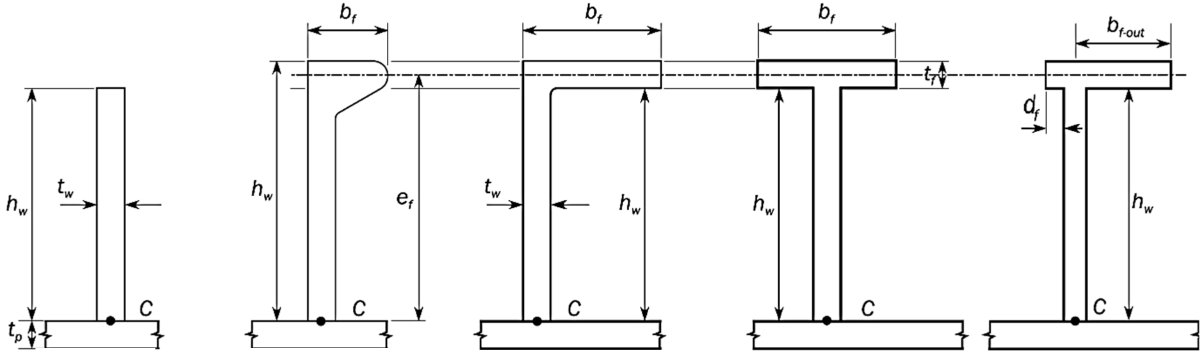
Amended	Remarks
RULES FOR THE SURVEY AND CONSTRUCTION OF STEEL SHIPS **Part C HULL CONSTRUCTION AND EQUIPMENT** **Part 1 GENERAL HULL REQUIREMENTS** **Annex 14.6 BUCKLING STRENGTH ASSESSMENT Of SHIP STRUCTURAL ELEMENTS** **Symbols** *EPP* : Elementary Plate Panel, as specified in **An1.2.3.1** PSM : Primary Supporting Member SP : Stiffened Panel, as specified in **An1.2.3.3** UP : Unstiffened Panel, as specified in **An1.2.3.3**	UR S35, Section1, Symbols
An1 Application and Definitions **An1.1 Application** **An1.1.1 General** **An1.1.1.1** This annex specifies a general buckling assessment procedure and is to be applied in conjunction with relevant requirements.	UR S35, Section1, 1.1.1

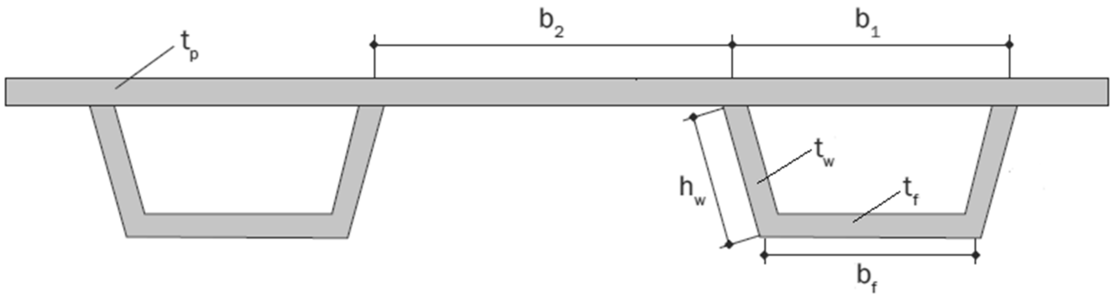
Amended-Original Requirements Comparison Table (Buckling Strength Assessments of Ship Structural Elements)

Amended	Remarks
An1.1.2 Overview **An1.1.2.1** The buckling checks are to be performed according to **An1:** General definitions regarding buckling capacity, allowable buckling utilisation factors and buckling check criteria. **An2:** The slenderness requirements of longitudinal and transverse stiffeners. **An3:** The prescriptive buckling requirements of plates, longitudinal and transverse stiffeners, primary supporting members and other structures subject to hull girder stresses. **An4:** Direct strength analysis (usually by finite element method) buckling requirements of hatch cover structural members including plates, stiffeners and primary supporting members. **An5:** The determination of buckling capacities of plate panels, stiffeners, primary supporting members and column structures.	UR S35, Section1, 1.2.1
An1.1.2.2 Buckling Assessment For the buckling assessment of a ship hull girder, a hatch cover or some structural component, the slenderness requirements as specified in **An2**, and the buckling requirements as specified in **An3** or **An4** are to be checked as per the requirements of the applicable relevant requirements.	UR S35, Section1, 1.2.2
An1.1.2.3 Alternative Methods This annex 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 annex, 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.	UR S35, Section1, 1.2.3

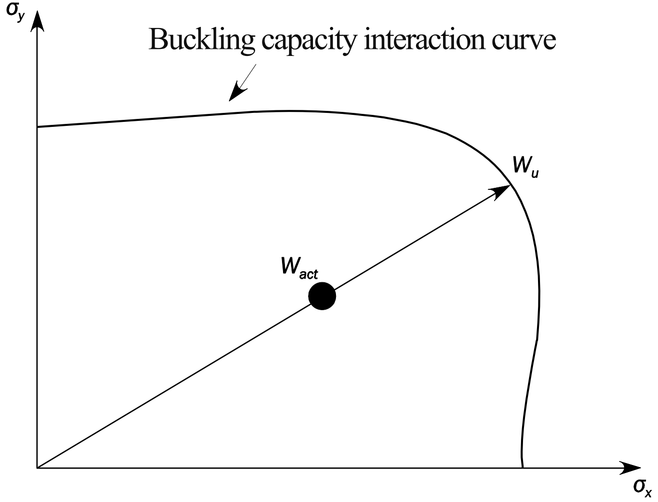
Amended	Remarks
An1.2 Terms and Assumptions **An1.2.1 Buckling** **An1.2.1.1 Buckling Strength** Buckling strength or capacity refers to the strength of a structure under in-plane compressions and/or shear and lateral load. Buckling strength with consideration of the buckling behavior in **An1.2.1.2** 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.	UR S35, Section1, 2.1.1
An1.2.1.2 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 utilised. 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.	UR S35, Section1, 2.1.2
An1.2.2 Net Scantling Approach **An1.2.2.1 General** Unless otherwise specified, all the scantling requirements, including slenderness requirements, in this annex are based on net scantlings obtained by removing full corrosion addition t_c from the gross offered thicknesses.	UR S35, Section1, 2.2.1
An1.2.2.2 Corrosion Addition Corrosion addition t_c referred in this annex is defined in the relevant requirements.	UR S35, Section1, 2.2.2
An1.2.2.3 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 requirements.	UR S35, Section1, 2.2.3

Amended	Remarks
An1.2.3 Structural Idealisation **An1.2.3.1 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 **Fig. An1**.	UR S35, Section1, 2.3.1
Fig. An1 Elementary Plate Panel (*EPP*) Definition 	
An1.2.3.2 Standard Types of Stiffeners Definitions of the cross-sectional dimensions of typical stiffener types are shown in **Fig. An2**, which are flat bars, bulb flats, angles, L2 and T bars. If applicable, other types of stiffeners can be idealised to one of the typical types in **Fig. An2** 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 **Fig. An3**. Unless otherwise specified, the full span or full length ℓ (*mm*) of a stiffener is to be used for buckling check, which equals to the spacing between primary supporting members. Symbolic dimensions of the cross-sections are as below: b_1 : Width (*mm*) of the attached plate enclosed by the U-type stiffener, as shown in **Fig. An3**.	UR S35, Section1, 2.3.2

Amended	Remarks
b_2 : Width (*mm*) of the attached plate between adjacent U-type stiffeners, as shown in **Fig. An3**. b_f : Width (*mm*) of the flange or face plate of the stiffener, as shown in **Fig. An2** and **Fig. An3**. b_{f-out}: Maximum distance (*mm*) from mid thickness (*mm*) of the web to the flange edge, as shown in **Fig. An2**. d_f: Breadth (*mm*) of the extended part of the flange for L2 profiles, as shown in **Fig. An2**. e_f: Distance (*mm*) from attached plating to centre of flange, as shown in **Fig. An2**. For its detailed definition, refer to **An5 Symbols**. h_w: Depth (*mm*) of stiffener web, as shown in **Fig. An2** and **Fig. An3**. t_f: Net flange thickness (*mm*). t_p: Net thickness of plate (*mm*). t_w: Net web thickness (*mm*).	
Fig. An2 Dimensions of Typical Stiffener Cross Sections  Flat bars Bulb flats Angles T bars L2 	UR S35, Section1, Figure 3

Amended	Remarks
Fig. An3 Dimensions of a U-type Stiffener Cross Section 	UR S35, Section1, Figure 4
An1.2.3.3 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*).	UR S35, Section1, 2.3.3
An1.2.4 Sign Convention **An1.2.4.1 Stresses** In this annex, compressive and shear stresses are to be taken as positive, tension stresses are to be taken as negative.	UR S35, Section1, 2.4.1

Amended	Remarks
An1.3 Assessment Methods and Acceptance Criteria **An1.3.1 Assessment Methods** **An1.3.1.1 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.	UR S35, Section1, 3.1.1
An1.3.1.2 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 **An1.2.3.3**, 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	UR S35, Section1, 3.1.2
An1.3.2 Buckling Utilisation Factor **An1.3.2.1** The utilisation factor, η, is defined as the ratio between the applied loads and the corresponding buckling capacity.	UR S35, Section1, 3.2.1

Amended	Remarks
An1.3.2.2 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 **Fig. An4**, and is to be taken as: $\eta_{act} = \frac{W_{act}}{W_U} = \frac{1}{\gamma_c}$ Where: W_{act}: Equivalent applied stress. The actual applied stresses are given in **An3** and **An4** 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 **An5**. 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. **Fig. An4** illustrates the buckling capacity and the buckling utilisation factor of a structural member subject to σ_x and σ_y stresses.	UR S35, Section1, 3.2.2
Fig. An4 Example of Buckling Capacity and Buckling Utilisation Factor  	UR S35, Section1, Figure 5

Amended-Original Requirements Comparison Table (Buckling Strength Assessments of Ship Structural Elements)

Amended	Remarks
An1.3.3 Allowable Buckling Utilisation Factor **An1.3.3.1** The allowable buckling utilisation factor η_{all} is to be taken according to the relevant requirements.	UR S35, Section1, 3.3.1
An1.3.4 Buckling Acceptance Criteria **An1.3.4.1** 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 **An1.3.2.2**. η_{all}: Allowable buckling utilisation factor as defined in **An1.3.3.1**.	UR S35, Section1, 3.4.1

Amended	Remarks
Symbols For symbols not defined in this section, refer to **An1.2.3.2**. σ_Y : Specified minimum yield stress (N/mm^2) of the structural member being considered	UR S35, Section2, Symbols
An2 Slenderness Requirements **An2.1 General** **An2.1.1 Introduction** **An2.1.1.1** The stiffener elements except for U-type stiffeners are to comply with the applicable slenderness and proportion requirements given in **An2**.	UR S35, Section2, 1.1.1
An2.2Stiffeners **An2.2.1 Proportions of Stiffeners** **An2.2.1.1 Net Thickness of All Stiffener Types** The net thickness of stiffeners is to satisfy the following criteria: (a) Stiffener web plate: $t_w \geq \frac{h_w}{C_w}\sqrt{\frac{\sigma_Y}{235}}$ (b) Flange: $t_f \geq \frac{b_{f-out}}{C_f}\sqrt{\frac{\sigma_Y}{235}}$ C_w, C_f: Slenderness coefficients given in **Table An1** If requirement **(b)** is not fulfilled, the effective free flange outstand (mm) used in strength assessment including the calculation of actual net section modulus, is to be taken as:	UR S35, Section2, 2.1.1

Amended	Remarks
$b_{f-out-max} = C_f t_f \sqrt{\frac{235}{\sigma_Y}}$	
Table An1 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 / - 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 An1**, and the edge stiffener can be assessed as a flat bar stiffener according to **An2.2.1.1**. The requirement to flange in **An2.2.1.2** is to apply.	UR S35, Section2, Table 1
An2.2.1.2 Net Dimensions of Angle and T-Bars The total flange breadth b_f (mm) for angle and T-bars is to satisfy the following criterion: $b_f \geq 0.2h_w$	UR S35, Section2, 2.1.2
An2.3 Primary Supporting Members **An2.3.1 Proportions and Stiffness** **An2.3.1.1 Proportions of Web Plate and Flange** The scantlings of webs and flanges of primary supporting members are to comply with the relevant requirements.	UR S35, Section2, 3.1.1

Table An1 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	-

Amended	Remarks
Symbols η_{all}: Allowable buckling utilisation factor, as defined in **An1.3.3.1** *LCP*: Load Calculation Point as defined in **An3.1.2.1**	UR S35, Section3, Symbols
An3 Buckling Requirements for Hull Girder **An3.1 General** **An3.1.1 Introduction** **An3.1.1.1** 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.	UR S35, Section3, 1.1.1
An3.1.1.2 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 requirements.	UR S35, Section3, 1.1.2
An3.1.1.3 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 requirements with 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 **An3.1.2.1**, and is to be applied together with the hull girder stress combinations defined in the relevant requirements.	UR S35, Section3, 1.1.3

Amended	Remarks
An3.1.2 Definitions **An3.1.2.1 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 An2**. (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. $P = \frac{\ell_1}{\ell}\frac{p_L}{2}$ when the upper end of the vertical stiffener is at or above the lowest zero pressure level, see **Fig. An6**. Where: ℓ_1: Distance (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.	UR S35, Section3, 1.2.1

Amended-Original Requirements Comparison Table (Buckling Strength Assessments of Ship Structural Elements)

Amended	Remarks

Table An2 Load Calculation Points (*LCP*) Coordinates for Plate Buckling Assessment

UR S35, Section3, Table 1

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 **Fig. An5**)	Outboard and inboard ends of the *EPP* (points $A1$ and $A2$ in **Fig. An5**)	Mid-point of *EPP* (point B in **Fig. An5**)
z coordinate	Corresponding to x and y values		

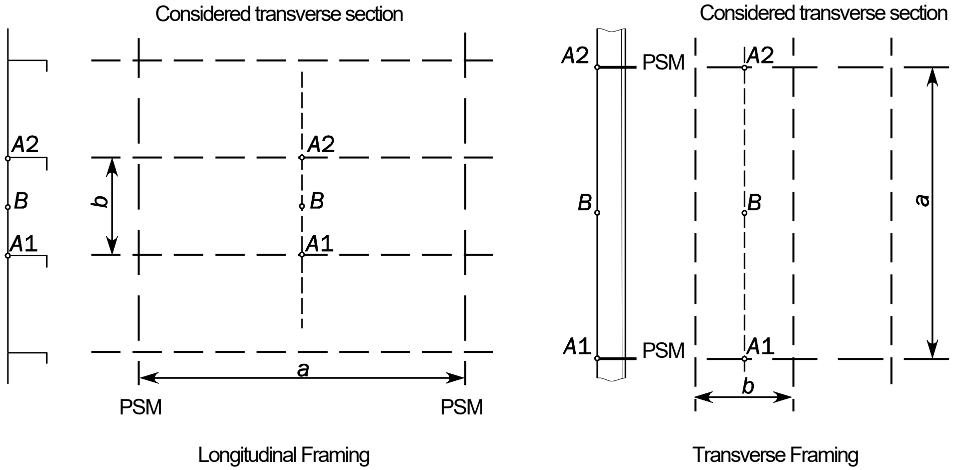
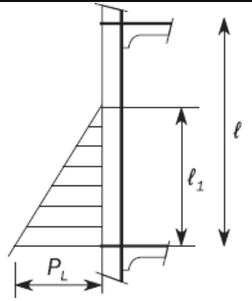


Fig. An5 *LCP* for Plate Buckling Assessment

UR S35, Section3, Figure 1

Fig. An6 Definition of Pressure for Vertical Stiffeners



UR S35, Section3, Figure 2

An3.1.3 Assumptions for Equivalent Plate Panels

UR S35, Section3, 1.3.1

An3.1.3.1 Longitudinal Stiffening with Varying Plate Thickness

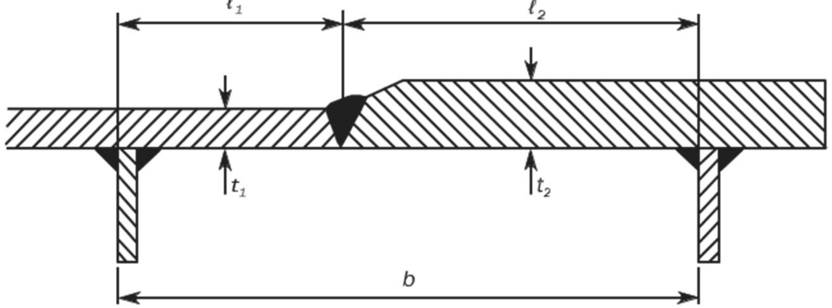
In longitudinal stiffening arrangement, when the plate thickness varies over the width b, of a plate panel, the buckling check is to be performed for an equivalent plate panel width, combined with the smaller plate thickness, t_1. The width (*mm*) of this equivalent plate panel, b_{eq} is defined by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

$$b_{eq} = \ell_1 + \ell_2 \left(\frac{t_1}{t_2}\right)^{1.5}$$

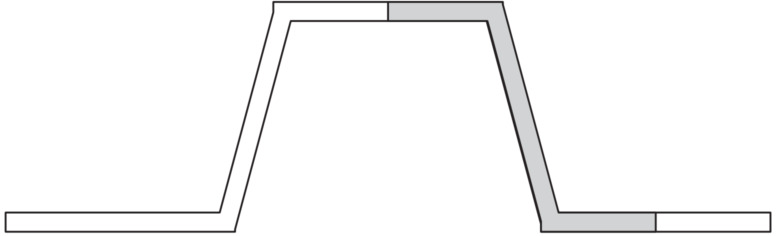
Where:

ℓ_1 : Width of the part of the plate panel with the smaller plate thickness, t_1 (*mm*) as defined in **Fig. An7**.

ℓ_2 : Width of the part of the plate panel with the greater plate thickness, t_2 (*mm*) as defined in **Fig. An7**.

Amended	Remarks
Fig. An7 Plate Thickness Change Over the Width  	UR S35, Section3, Figure 3
An3.1.3.2 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*.	UR S35, Section3, 1.3.2
An3.1.3.3 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.	UR S35, Section3, 1.3.3
An3.2 Buckling Criteria **An3.2.1 Overall Stiffened Panel** **An3.2.1.1** The buckling strength of overall stiffened panels is to satisfy the following criterion: $\eta_{overall} \leq \eta_{all}$ $\eta_{overall}$: Maximum overall buckling utilisation factor as defined **An5.2.1**.	UR S35, Section3, 2.1.1

Amended	Remarks
An3.2.2 Plates **An3.2.2.1** 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 **An5.2.2** 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 An5** corresponding to the shorter edge of the plate panel clamped are to be considered together with a mean σ_y stress and $\psi_y = 1$.	UR S35, Section3, 2.2.1
An3.2.3 Stiffeners **An3.2.3.1** 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 **An5.2.3**. Note 1: This buckling check can only be fulfilled when the overall stiffened panel buckling check, as defined in **An3.2.1**, 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.	UR S35, Section3, 2.3.1
An3.2.4 Vertically Corrugated Longitudinal Bulkheads **An3.2.4.1** The shear buckling strength of vertically corrugated longitudinal bulkheads is to satisfy the following criterion: $\eta_{shear} \leq \eta_{all}$ η_{shear}: Maximum shear buckling utilisation factor defined as $\eta_{shear} = \frac{\tau_{bhd}}{\tau_c}$ τ_{bhd}: Shear stress (N/mm^2) in the bulkhead taken as the hull girder shear stress defined in the relevant requirements.	UR S35, Section3, 2.4.1

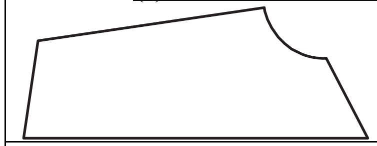
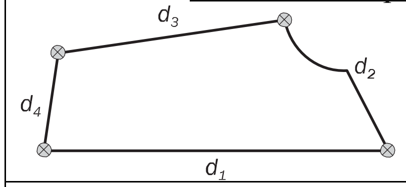
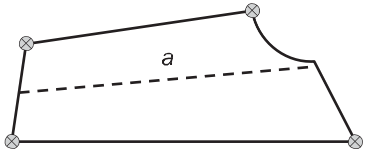
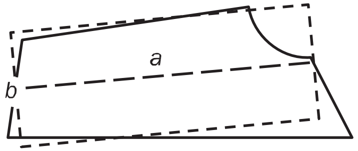
Amended	Remarks
τ_c: Shear critical stress (*N/mm²*) as defined in **An5.2.2.3**.	
An3.2.5 Horizontally Corrugated Longitudinal Bulkheads **An3.2.5.1** Each corrugation unit within the extension of half flange, web and half flange (i.e. single corrugation as shown in grey in **Fig. An8**) is to satisfy the following criterion: $\eta_{column} \leq \eta_{all}$ Where: η_{column}: Overall column buckling utilisation factor, as defined in **An5.3.1**.	UR S35, Section3, 2.5.1
Fig. An8 Single Corrugation 	UR S35, Section3, Figure 4

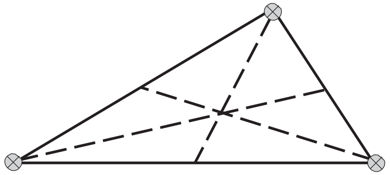
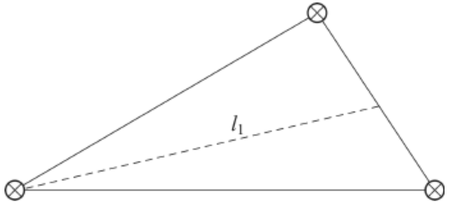
Amended-Original Requirements Comparison Table (Buckling Strength Assessments of Ship Structural Elements)

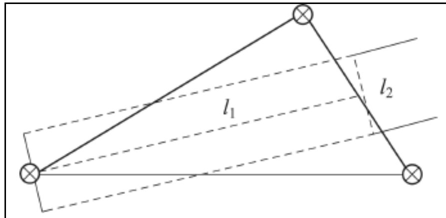
Amended	Remarks
Symbols σ_{Y_P}: Yield stress of the plate panel, as defined in **An4.2.1.3** σ_{Y_S}: Yield stress of the stiffener, as defined in **An4.2.1.3** α: Aspect ratio of the plate panel, as defined in the symbol list of **An5** η_{all}: Allowable buckling utilisation factor, as defined in **An1.3.3.1**	UR S35, Section4, Symbols
An4 Buckling Requirements for Direct Strength Analysis of Hatch Covers **An4.1 General** **An4.1.1 Introduction** **An4.1.1.1** The requirements of this section apply for 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.	UR S35, Section4, 1.1.1
An4.1.1.2 All structural elements in the direct strength analysis carried out according to the relevant requirements 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	UR S35, Section4, 1.1.2

Amended	Remarks
An4.2 Stiffened and Unstiffened Panels **An4.2.1 General** **An4.2.1.1** 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 **An1.3.1.1** 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 **An1.3.1.2**.	UR S35, Section4, 2.1.1
An4.2.1.2 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.	UR S35, Section4, 2.1.2
An4.2.1.3 Yield Stress of the Plate Panel and Stiffener The panel yield stress σ_{Y_P} is taken as the minimum value of the specified yield stresses of the elements within the plate panel. The stiffener yield stress σ_{Y_S} is taken as the minimum value of the specified yield stresses of the elements within the stiffener.	UR S35, Section4, 2.1.3

Amended	Remarks
An4.2.2 Stiffened Panels **An4.2.2.1** For a stiffened panel (SP), each stiffener with attached plate is to be idealised as a stiffened panel model of the extent defined in the relevant requirements.	UR S35, Section4, 2.2.1
An4.2.2.2 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.	UR S35, Section4, 2.2.2
An4.2.2.3 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.	UR S35, Section4, 2.2.3
An4.2.3 Unstiffened Panels **An4.2.3.1 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 **An4.2.3.2** for irregular geometry and **An4.2.3.3** for triangular geometry and to comply with buckling assessment.	UR S35, Section4, 2.3.1
An4.2.3.2 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:	UR S35, Section4, 2.3.2

Amended	Remarks
(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.  	UR S35, Section4, 2.3.2
(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.  	UR S35, Section4, 2.3.2
(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.	UR S35, Section4, 2.3.2
(e) The length of shorter side, b (mm) is to be taken as: $b = \frac{A}{a}$ A: Area of the plate (mm^2) a: Length defined in **(d)** (mm)  	UR S35, Section4, 2.3.2

Amended	Remarks
(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.	UR S35, Section4, 2.3.2
An4.2.3.3 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. 	UR S35, Section4, 2.3.3
(b) The longest median is identified. This median the length of which is ℓ_1 (*mm*) defines the longitudinal direction for the capacity model. 	UR S35, Section4, 2.3.3
(c) The width of the model, ℓ_2 (*mm*) is to be taken as: $\ell_2 = \frac{A}{\ell_1}$ Where: A: Area of the plate (mm^2)	UR S35, Section4, 2.3.3

Amended	Remarks
	
(d) The lengths of shorter side, b (*mm*), and of the longer side, a (*mm*) of the equivalent rectangular plate panel are to be taken as: $b = \frac{\ell_2}{C_{tri}}$ $a = \ell_1 C_{tri}$ Where: $C_{tri} = 0.4\frac{\ell_2}{\ell_1} + 0.6$	UR S35, Section4, 2.3.3
(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.	UR S35, Section4, 2.3.3
An4.2.4 Reference Stress **An4.2.4.1** The stress distribution is to be taken from the direct strength analysis according to the relevant requirements and applied to the buckling model.	UR S35, Section4, 2.4.1
An4.2.4.2 For FE analysis, the reference stresses are to be calculated using the stress based reference stresses as defined in **Appendix 14.6「STRESS BASED REFERENCE STRESSES」**.	UR S35, Section4, 2.4.2

Amended	Remarks
An4.2.5 Lateral Pressure **An4.2.5.1** The lateral pressure applied to the direct strength analysis is also to be applied to the buckling assessment.	UR S35, Section4, 2.5.1
An4.2.5.2 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 (*mm*^{2}) P_i: Lateral pressure of the *i*-th plate element (*N/mm*^{2}) n: Number of finite elements in the buckling panel	UR S35, Section4, 2.5.2
An4.2.6 Buckling Criteria **An4.2.6.1 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 **An5.2.2** where UP-A model is to be used.	UR S35, Section4, 2.6.1
An4.2.6.2 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 **An5.2.2** where UP-B model is to be used.	UR S35, Section4, 2.6.2

Amended-Original Requirements Comparison Table (Buckling Strength Assessments of Ship Structural Elements)

Amended	Remarks
An4.2.6.3 SP-A The compressive buckling strength of SP-A is to satisfy the following criterion: $\eta_{SP-A} \le \eta_{all}$ 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 **An5.2.1**. ・ The plate buckling utilisation factor η_{plate} as defined in **An5.2.2** where SP-A model is to be used. ・ The stiffener buckling 27tilization factor $\eta_{stiffener}$ as defined in **An5.2.3** considering separately the properties (thickness, dimensions), the pressures defined in **An4.2.5.2** 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 **An5.2.1** is satisfied.	UR S35, Section4, 2.6.3
An4.2.6.4 SP-B The compressive buckling strength of SP-B is to satisfy the following criterion: $\eta_{SP-B} \le \eta_{all}$ 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 **An5.2.1**. ・ The plate buckling utilisation factor η_{plate} as defined in **An5.2.2** where SP-B model is to be used. ・ The stiffener buckling utilisation factor $\eta_{stiffener}$ as defined in **An5.2.3** considering separately the properties (thickness, dimensions), the pressures defined in **An4.2.5.2** 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 **An5.2.1**, is satisfied.	UR S35, Section4, 2.6.4

Amended	Remarks
An4.2.6.5 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 **An1.3.2.2** and the stress multiplier factor at failure γ_c which can be calculated following the requirements in **An5.2.4**.	UR S35, Section4, 2.6.5

Amended	Remarks
Symbols A_p: Net sectional area (mm^2) of the stiffener attached plating taken as: $A_p = st_p$ A_s: Net sectional area (mm^2) of the stiffener without attached plating a: Length (*mm*) of the longer side of the plate panel b: Length (*mm*) of the shorter side of the plate panel b_{eff}: Effective width (*mm*) of the attached plating of a stiffener as defined in **An5.2.3.5** b_{eff1}: Effective width (*mm*) of the attached plating of a stiffener 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 (*mm*) of the stiffener flange b_1, b_2: Width (*mm*) of plate panel on each side of the considered stiffener. For stiffened panels fitted with U-type stiffeners, b_1 and b_2 are as defined in **Fig. An3**. C_{x1}, C_{x2}: Reduction factor defined in **Table An5** calculated for the *EPP*1 and *EPP*2 on each side of the considered stiffener according to case 1 d: Length (*mm*) of the side parallel to the cylindrical axis of the cylinder corresponding to the curved plate panel as shown in **Table An6** d_f: Breadth (*mm*) of the extended part of the flange for L2 profiles as shown in **Fig. An2** e_f: Distance (*mm*) from attached plating to centre of flange as shown in **Fig. An2** 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 **An5.2.2.4**	UR S35, Section5, Symbols

Amended	Remarks
F_{tran}: Coefficient defined in **An5.2.2.5** h_w: Depth (*mm*) of stiffener web as shown in **Fig. An2** ℓ: Span (*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 (*mm*) of curved plate panel σ_{Y_P}: Specified minimum yield stress (N/mm^2) of the plate σ_{Y_S}: Specified minimum yield stress (N/mm^2) of the stiffener S: Partial safety factor, unless otherwise specified in the relevant requirements, to be taken as 1.0 t_p: Net thickness (*mm*) of plate panel t_w: Net stiffener web thickness (*mm*) t_f: Net flange thickness (*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 An5** 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 (N/mm^2) applied on the edge along *x*-axis of the buckling panel σ_y: Normal stress (N/mm^2) applied on the edge along *y*-axis of the buckling panel σ_1: Maximum normal stress (N/mm^2) along a panel edge σ_2: Minimum normal stress (N/mm^2) along a panel edge σ_E: Elastic buckling reference stress (N/mm^2) to be taken as: ・ For the application of the limit state of plane plate panels according to **An5.2.2.1**: $\sigma_E = \frac{\pi^2 E}{12(1-\nu^2)}\left(\frac{t_p}{b}\right)^2$	

Amended	Remarks
・ For the application of the limit state of curved plate panels according to **An5.2.2.6**: $\sigma_E = \frac{\pi^2 E}{12(1-\nu^2)}\left(\frac{t_p}{d}\right)^2$ τ: Applied shear stress (*N/mm*^{2}) τ_c: Buckling strength in shear (*N/mm*^{2}) as defined in **An5.2.2.3** 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	
An5 Buckling Capacity **An5.1 General** **An5.1.1 Introduction** **An5.1.1.1** This section contains the methods for determination of the buckling capacities of plate panels, stiffeners, primary supporting members, and columns.	UR S35, Section5, 1.1.1
An5.1.1.2 For the application of this section, the stresses σ_x, σ_y and τ applied on the structural members are defined in: ・ **An3** for hull girder prescriptive buckling requirements. ・ **An4** for direct strength analysis buckling requirements of hatch covers.	UR S35, Section5, 1.1.2

Amended	Remarks
An5.1.1.3 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 **An5.2.1.1**, **An5.2.2.1** and **An5.2.3.4** are equal to 1.0, respectively.	UR S35, Section5, 1.1.3
An5.1.1.4 Buckling Utilisation Factor The buckling utilisation factor of the structural member is equal to the highest utilisation factor obtained for the different buckling modes.	UR S35, Section5, 1.1.4
An5.1.1.5 Lateral Pressure The lateral pressure is to be applied and considered as constant for the calculation of buckling capacities as defined in **An5.1.1.3**.	UR S35, Section5, 1.1.5
An5.2 Buckling Capacity of Plate Panels **An5.2.1 Overall Stiffened Panels** **An5.2.1.1** 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 **An5.2.3.4**: $\frac{\gamma_c}{\gamma_{GEB}} = 1$ with corresponding buckling utilisation 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: $\gamma_{GEB} = \gamma_{GEB,bi+\tau}$ for $\tau \neq 0$ and ($\sigma_x > 0$ or $\sigma_y > 0$) $\gamma_{GEB} = \gamma_{GEB,bi}$ for $\tau = 0$ and ($\sigma_x > 0$ or $\sigma_y > 0$) $\gamma_{GEB} = \gamma_{GEB,\tau}$ for $\tau \neq 0$ and ($\sigma_x \leq 0$ and $\sigma_y \leq 0$) where $\gamma_{GEB,bi}$, $\gamma_{GEB,\tau}$ and $\gamma_{GEB,bi+\tau}$ are stress multiplier factors of the global elastic buckling capacity for different load	UR S35, Section5, 2.1.1

Amended	Remarks
combinations as defined in **An5.2.1.2**, **An5.2.1.3** and **An5.2.1.4**, respectively. For the calculation of $\gamma_{GEB,bi}$, $\gamma_{GEB,\tau}$ and $\gamma_{GEB,bi+\tau}$, neither σ_x nor σ_y are to be taken less than 0. σ_x, σ_y: Applied normal stress (N/mm^2) to the plate panel to be taken as defined in **An5.2.2.7**. τ: Applied shear stress (N/mm^2) to be taken as defined in **An5.2.2.7**.	
An5.2.1.2 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 (N/mm) of the stiffened panel 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 **Fig. An3**. N_y: Load per unit length applied on the edge along y-axis (N/mm) of the stiffened panel taken as $N_y = c\sigma_y t_p$ L_{B1}: Stiffener span (mm) distance between primary supporting members, i.e. $L_{B1} = \ell$. Specially, for vertically stiffened side shell of single side skin bulk carriers, $L_{B1} = 0.8\ell$. L_{B2}: Total width (mm) of stiffened panel between lateral supports taken as 6 times of the stiffener spacing, i.e. 6s. n: Number of half waves along the direction perpendicular to the stiffener axis. The factor $\gamma_{GEB,bi}$ is to be minimised 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: $c = 0.5(1+\Psi)$ for $0 \leq \Psi \leq 1$ $c = \frac{1}{2(1-\Psi)}$ for $\Psi < 0$ Ψ: Edge stress ratio for case 2 according to **Table An5**	UR S35, Section5, 2.1.2

Amended	Remarks
$\sigma_{x,av}$: Average stress for both plate and stiffener, taken as: $\sigma_{x,av} = \sigma_x - \nu c \sigma_y A_s / \left(A_p + A_s\right) \geq 0$ for $\sigma_x > 0$ and $\sigma_y > 0$ $\sigma_{x,av} = \sigma_x$ for $\sigma_x \leq 0$ or $\sigma_y \leq 0$ $D_{11}, D_{12}, D_{22}, D_{33}$: Bending stiffness coefficients (*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: Breadth of U-type stiffener web as defined in **Fig. An3**. I_{eff}: Moment of inertia (*cm*^{4}) of the stiffener including effective width of attached plating, the same as I defined in **An5.2.3.4**.	
An5.2.1.3 The stress multiplier factor $\gamma_{GEB,\tau}$ for the stiffened panel subjected to pure shear load is taken as: $\gamma_{GEB,\tau} = \frac{\sqrt[4]{D_{11}^3 D_{22}}}{\left(L_{B1}/2\right)^2 N_{xy}} \left[8.125 + 5.64\sqrt{\frac{(D_{12}+D_{33})^2}{D_{11}D_{22}}} - 0.6\frac{(D_{12}+D_{33})^2}{D_{11}D_{22}}\right]$ for $D_{11}D_{22} \geq (D_{12}+D_{33})^2$ $\gamma_{GEB,\tau} = \frac{\sqrt{2D_{11}(D_{12}+D_{33})}}{\left(L_{B1}/2\right)^2 N_{xy}} \left[8.3 + 1.525\frac{D_{11}D_{22}}{(D_{12}+D_{33})^2} - 0.493\frac{D_{11}^2 D_{22}^2}{(D_{12}+D_{33})^4}\right]$ for $D_{11}D_{22} < (D_{12}+D_{33})^2$ Where:	UR S35, Section5, 2.1.3

Amended	Remarks
$N_{xy} = \tau t_p$	
An5.2.1.4 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 **An5.2.1.2** and **An5.2.1.3**, respectively.	UR S35, Section5, 2.1.4
An5.2.2 Plates **An5.2.2.1 Plate Limit State** The plate limit state is based on the following interaction formulae: $\left(\frac{\gamma_{c1}\sigma_x S}{\sigma_{cx}}\right)^{e_0} - B\left(\frac{\gamma_{c1}\sigma_x S}{\sigma_{cx}}\right)^{\frac{e_0}{2}}\left(\frac{\gamma_{c1}\sigma_y S}{\sigma_{cy}}\right)^{\frac{e_0}{2}} + \left(\frac{\gamma_{c1}\sigma_y S}{\sigma_{cy}}\right)^{e_0} + \left(\frac{\gamma_{c1}\lvert\tau\rvert S}{\tau_c}\right)^{e_0} = 1$ $\left(\frac{\gamma_{c2}\sigma_x S}{\sigma_{cx}}\right)^{2/\beta_p^{0.25}} + \left(\frac{\gamma_{c2}\lvert\tau\rvert S}{\tau_c}\right)^{2/\beta_p^{0.25}} = 1$ for $\sigma_x \geq 0$ $\left(\frac{\gamma_{c3}\sigma_y S}{\sigma_{cy}}\right)^{2/\beta_p^{0.25}} + \left(\frac{\gamma_{c3}\lvert\tau\rvert S}{\tau_c}\right)^{2/\beta_p^{0.25}} = 1$ for $\sigma_y \geq 0$ $\frac{\gamma_{c4}\lvert\tau\rvert S}{\tau_c} = 1$ with $\gamma_c = Min(\gamma_{c1}, \gamma_{c2}, \gamma_{c3}, \gamma_{c4})$ and corresponding buckling utilisation factor defined as $\eta_{plate} = \frac{1}{\gamma_c}$ Where:	UR S35, Section5, 2.2.1

Amended	Remarks

σ_x, σ_y: Applied normal stress (N/mm^2) to the plate panel to be taken as defined in **An5.2.2.7**.

τ: Applied shear stress (N/mm^2) to the plate panel.

σ_{cx}: Ultimate buckling stress (N/mm^2) in direction parallel to the longer edge of the buckling panel as defined in **An5.2.2.3**.

σ_{cy}: Ultimate buckling stress (N/mm^2) in direction parallel to the shorter edge of the buckling panel as defined in **An5.2.2.3**.

τ_c: Ultimate buckling shear stress (N/mm^2) as defined in **An5.2.2.3**.

$\gamma_{c1}, \gamma_{c2}, \gamma_{c3}, \gamma_{c4}$: 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 An3**.

e_0: Coefficient given in **Table An3**.

β_p: Plate slenderness parameter taken as:

$$\beta_p = \frac{b}{t_p}\sqrt{\frac{\sigma_{Y\_P}}{E}}$$

Table An3 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

UR S35, Section5, Table 1

An5.2.2.2 Reference Degree of Slenderness

The reference degree of slenderness is to be taken as:

$$\lambda = \sqrt{\frac{\sigma_{Y\_P}}{K\sigma_E}}$$

Where:

K: Buckling factor, as defined in **Table An5** and **Table An6**.

UR S35, Section5, 2.2.2

UR S35, Section5, 2.2.3

An5.2.2.3 Ultimate Buckling Stresses

The ultimate buckling stresses (*N/mm²*) of plate panels are to be taken as:

$$\sigma_{cx} = C_x \sigma_{Y\_P}$$

$$\sigma_{cy} = C_y \sigma_{Y\_P}$$

The ultimate buckling stress of plate panels subject to shear (*N/mm²*) is to be taken as:

$$\tau_c = C_\tau \frac{\sigma_{Y\_P}}{\sqrt{3}}$$

Where:

C_x, C_y, C_τ : Reduction factors, as defined in **Table An5**

- For the 1st Equation of **An2.2.1**, 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 An5** 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 An5** by using

$$c_1 = 1$$

  - For vertically stiffened single side skin of bulk carrier, C_y is calculated according to **Table An5** by using

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

  - For corrugation of corrugated bulkheads, C_y is calculated according to **Table An5** 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 An5**. If the boundary conditions differ significantly from simple support, a more appropriate boundary condition can be applied according to the different cases of **Table An5** subject to the agreement of the Society.

Amended	Remarks
An5.2.2.4 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 An4**. 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 An4**, the value of c is to be agreed by the Society. In such a case, value of c higher than those mentioned in **Table An4** can be used, provided it is verified by buckling strength check of panel using non-linear FE analysis and deemed appropriate by the Society.	UR S35, Section5, 2.2.4

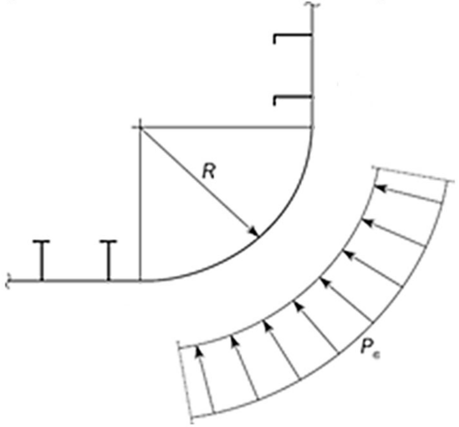
Amended-Original Requirements Comparison Table (Buckling Strength Assessments of Ship Structural Elements)

Amended	Remarks
Table An4 Correction Factor F_{long}	UR S35, Section5, Table 2

Structural element types			F_{long}	c
Unstiffened Panel			1.0	N/A
Stiffened Panel	Stiffener not fixed at both ends		1.0	N/A
	Stiffener fixed at both ends	Flat bar(1)	$F_{long} = c + 1 \; for \; \frac{t_w}{t_p} > 1$ $F_{long} = c\left(\frac{t_w}{t_p}\right)^3 + 1 \; for \; \frac{t_w}{t_p} \le 1$	0.10
		Bulb profile		0.30
		Angle and L2 profiles		0.40
		T profile		0.30
		Girder of high rigidity (e.g. bottom transverse)	1.4	N/A
		U-type profile fitted on hatch cover(2)	- 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: $F_{long} = 1$	0.2

(1) t_w is the net web thickness (*mm*) without the correction defined in **An.5.2.3.2.**
(2) b_1, b_2 and t_w are defined in **Fig. An3.**

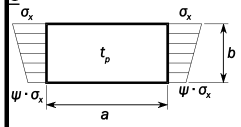
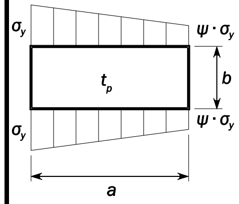
Amended	Remarks
An5.2.2.5 Correction Factor F_{tran} The correction factor F_{tran} is to be taken as: ・For transversely framed *EPP* of single side skin bulk carrier, between the hopper and top wing tank: ・F_{tran}=1.25 when the two adjacent frames are supported by one tripping bracket fitted in way 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. ・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: $F_{tran0} = Min\left(\frac{b_2}{b_1} + \frac{6b_2^2}{\pi^2 h_w (b_1+b_2)}\left(\frac{t_w}{t_p}\right)^3, 6\right)$ for *EPP* b_2 $F_{tran0} = Min\left(\frac{b_1}{b_2} + \frac{6b_1^2}{\pi^2 h_w (b_2+b_1)}\left(\frac{t_w}{t_p}\right)^3, 6\right)$ for *EPP* b_1 with b_1, b_2 and h_w as defined in **Fig. An3**. Coefficient F defined in case 2 of **Table An5** 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.	UR S35, Section5, 2.2.5
An5.2.2.6 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 **An5.2.2.1** is applicable. The curved plate limit state is based on the following interaction formula: $\left(\frac{\gamma_c \sigma_{ax} S}{C_{ax}\sigma_{Y\_P}}\right)^{1.25} - 0.5\left(\frac{\gamma_c \sigma_{ax} S}{C_{ax}\sigma_{Y\_P}}\right)\left(\frac{\gamma_c \sigma_{tg} S}{C_{tg}\sigma_{Y\_P}}\right) + \left(\frac{\gamma_c \sigma_{tg} S}{C_{tg}\sigma_{Y\_P}}\right)^{1.25} + \left(\frac{\gamma_c \tau \sqrt{3} S}{C_{\tau}\sigma_{Y\_P}}\right)^2 = 1.0$ with corresponding buckling utilisation factor defined as $\eta_{curved\_plate} = \frac{1}{\gamma_c}$	UR S35, Section5, 2.2.6

Amended	Remarks
Where: σ_{ax}: Applied axial stress (*N/mm*^{2}) to the cylinder corresponding to the curved plate panel. In case of tensile axial stresses, $\sigma_{ax} = 0$. σ_{tg}: Applied tangential stress (*N/mm*^{2}) to the cylinder corresponding to the curved plate panel. 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 An6**. 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 **An5.2.2.1**.	
Fig. An9 Transverse Stiffened Bilge Plating  	UR S35, Section5, Figure 1

Amended-Original Requirements Comparison Table (Buckling Strength Assessments of Ship Structural Elements)

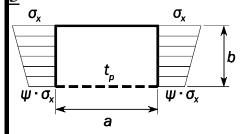
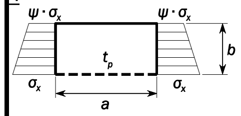
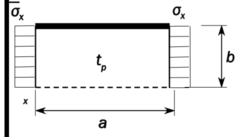
Amended	Remarks
Table An5 Buckling Factor and Reduction Factor for Plane Plate Panels	UR S35, Section5, Table 3

Table An5 Buckling Factor and Reduction Factor for Plane Plate Panels

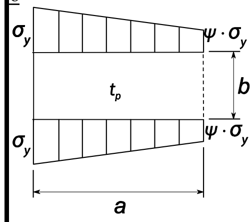
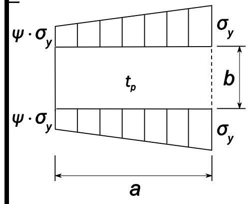
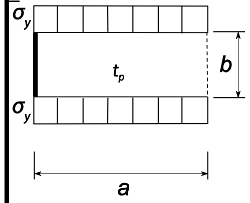
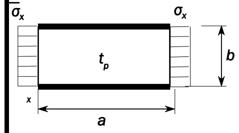
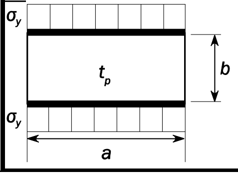
Case	Stress ratio ψ	Aspect ratio α	Buckling factor K	Reduction factor C
1 	$1 \geq \psi \geq 0$		$K_x = F_{long} \dfrac{8.4}{\psi + 1.1}$	When $\sigma_x \leq 0$: $C_x = 1$ When $\sigma_x > 0$: $C_x = 1$ for $\lambda \leq \lambda_c$ $C_x = c\left(\frac{1}{\lambda} - \frac{0.22}{\lambda^2}\right)$ for $\lambda > \lambda_c$ Where: $c = (1.25 - 0.12\psi) \leq 1.25$ $\lambda_c = \dfrac{c}{2}\left(1 + \sqrt{1 - \dfrac{0.88}{c}}\right)$
	$0 > \psi > -1$		$K_x = F_{long}[7.63 - \psi(6.26 - 10\psi)]$	
	$\psi \leq -1$		$K_x = F_{long}[5.975(1 - \psi)^2]$	
2 	$1 \geq \psi \geq 0$		$K_y = F_{tran} \dfrac{2\left(1 + \frac{1}{\alpha^2}\right)^2}{1 + \psi + \frac{(1-\psi)}{100}\left(\frac{2.4}{\alpha^2} + 6.9 f_1\right)}$	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: $c = (1.25 - 0.12\psi) \leq 1.25$ $R = \lambda(1 - \lambda/c)$ for $\lambda < \lambda_c$ $R = 0.22$ for $\lambda \geq \lambda_c$ $\lambda_c = 0.5c\left(1 + \sqrt{1 - 0.88/c}\right)$ $F = \left[1 - \left(\frac{K}{0.91} - 1\right)/\lambda_p^2\right] c_1 \geq 0$ $\lambda_p^2 = \lambda^2 - 0.5$ for $1 \leq \lambda_p^2 \leq 3$ C_1 as defined in **An5.2.2.3** $H = \lambda - \frac{2\lambda}{c(T + \sqrt{T^2 - 4})} \geq R$
		$\alpha \leq 6$	$f_1 = (1 - \psi)(\alpha - 1)$	
		$\alpha > 6$	$f_1 = 0.6(1 - \dfrac{6\psi}{\alpha})(\alpha + \dfrac{14}{\alpha})$ but not greater than $14.5 - \frac{0.35}{\alpha^2}$	
	$0 \geq \psi \geq 1 - \dfrac{4\alpha}{3}$		$K_y = \dfrac{200 F_{tran}(1 + \beta^2)^2}{(1 - f_3)(100 + 2.4\beta^2 + 6.9 f_1 + 23 f_2)}$	
		$\alpha > 6(1 - \psi)$	$f_1 = 0.6(\dfrac{1}{\beta} + 14\beta)$ but not greater than $14.5 - 0.35\beta^2$ $f_2 = f_3 = 0$	

Amended-Original Requirements Comparison Table (Buckling Strength Assessments of Ship Structural Elements)

Amended							Remarks
			$3(1-\psi) \leq \alpha \leq 6(1-\psi)$	$f_1 = \frac{1}{\beta} - 1$ $f_2 = f_3 = 0$	$T = \lambda + \frac{14}{15\lambda} + \frac{1}{3}$		
			$1.5(1-\psi) \leq \alpha < 3(1-\psi)$	$f_1 = \frac{1}{\beta} - (2-\omega\beta)^4 - 9(\omega\beta - 1)(\frac{2}{3} - \beta)$ $f_2 = f_3 = 0$			
		$0 \geq \psi \geq 1 - \frac{4\alpha}{3}$	$1 - \psi \leq \alpha < 1.5(1-\psi)$	For $\alpha > 1.5$: $f_1 = 2\left(\frac{1}{\beta} - 16(1 - \frac{\omega}{3})^4\right)\left(\frac{1}{\beta} - 1\right)$ $f_2 = 3\beta - 2$ $f_3 = 0$ For $\alpha \leq 1.5$: $f_1 = 2\left(\frac{1.5}{1-\psi} - 1\right)\left(\frac{1}{\beta} - 1\right)$ $f_2 = \frac{\psi(1-16f_4^2)}{1-\alpha}$ $f_3 = 0$ $f_4 = (1.5 - Min(1.5;\alpha))^2$			

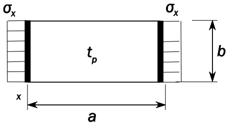
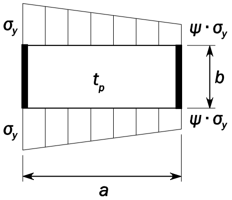
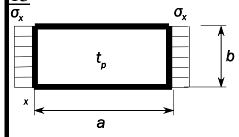
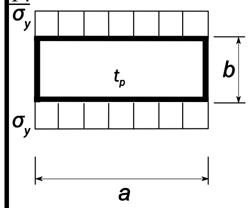
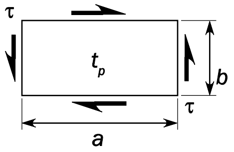
Amended							Remarks
			$0.75(1-\psi) \le \alpha < 1-\psi$	$f_1 = 0$ $f_2 = 1 + 2.31(\beta - 1) - 48\left(\frac{4}{3} - \beta\right) f_4^2$ $f_3 = 3f_4(\beta - 1)\left(\frac{f_4}{1.81} - \frac{\alpha - 1}{1.31}\right)$ $f_4 = (1.5 - Min(1.5; \alpha))^2$			
		$\psi < 1 - \frac{4\alpha}{3}$		$K_y = 5.972 F_{tran} \frac{\beta^2}{1 - f_3}$ Where: $f_3 = f_5\left(\frac{f_5}{1.81} + \frac{1 + 3\psi}{5.24}\right)$ $f_5 = \frac{9}{16}(1 + Max(-1; \psi))^2$			
	3 	$1 \ge \psi \ge 0$		$K_x = \frac{4(0.425 + 1/\alpha^2)}{3\psi + 1}$	ForUP-A: $C_x = 1$ for $\lambda \le 0.75$ $C_x = \frac{0.75}{\lambda}$ for $\lambda > 0.75$ ForUP-B: $C_x = 1$ for $\lambda \le 0.7$ $C_x = \frac{1}{\lambda^2 + 0.51}$ for $\lambda > 0.7$		
		$0 > \psi \ge -1$		$K_x = 4(0.425 + 1/\alpha^2)(1 + \psi) - 5\psi(1 - 3.42\psi)$			
	4 	$1 \ge \psi \ge -1$		$K_x = \left(0.425 + \frac{1}{\alpha^2}\right)\frac{3 - \psi}{2}$			
	5 		$\alpha \ge 1.64$	$K_x = 1.28$			
			$\alpha < 1.64$	$K_x = \frac{1}{\alpha^2} + 0.56 + 0.13\alpha^2$			

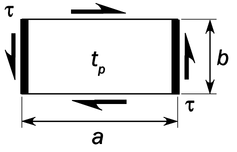
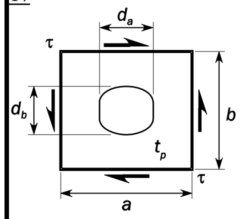
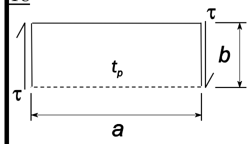
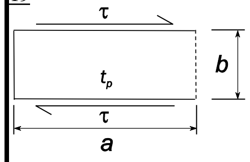
Amended-Original Requirements Comparison Table (Buckling Strength Assessments of Ship Structural Elements)

Amended						Remarks
	6 	$1 \geq \psi \geq 0$	$K_y = \frac{4(0.425 + \alpha^2)}{(3\psi + 1)\alpha^2}$	For UP-A: $C_x = 1$ for $\lambda \leq 0.75$ $C_x = \frac{0.75}{\lambda}$ for $\lambda > 0.75$ For UP-B: $C_y = 1$ for $\lambda \leq 0.7$ $C_y = \frac{1}{\lambda^2 + 0.51}$ for $\lambda > 0.7$		
		$0 > \psi \geq -1$	$K_y = 4(0.425 + \alpha^2)(1 + \psi)\frac{1}{\alpha^2}$ $-5\psi(1 - 3.42\psi)\frac{1}{\alpha^2}$			
	7 	$1 \geq \psi \geq -1$	$K_y = 4(0.425 + \alpha^2)\frac{(3 - \psi)}{2\alpha^2}$			
	8 	–	$K_y = 1 + \frac{0.56}{\alpha^2} + \frac{0.13}{\alpha^4}$			
	9 	–	$K_x = 6.97$	$C_x = 1$ for $\lambda \leq 0.83$ $C_x = 1.13\left(\frac{1}{\lambda} - \frac{0.22}{\lambda^2}\right)$ for $\lambda > 0.83$		
	10 	–	$K_y = 4 + \frac{2.07}{\alpha^2} + \frac{0.67}{\alpha^4}$	$C_y = 1$ for $\lambda \leq 0.83$ $C_y = 1.13\left(\frac{1}{\lambda} - \frac{0.22}{\lambda^2}\right)$ for $\lambda > 0.83$		

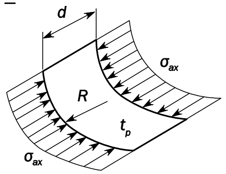
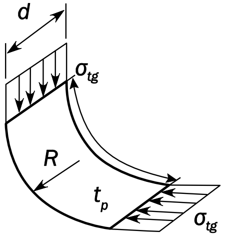
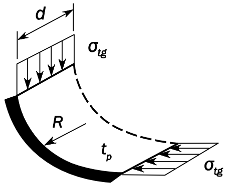
Amended-Original Requirements Comparison Table (Buckling Strength Assessments of Ship Structural Elements)

Amended	Remarks

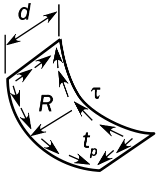
	11 	—	$\alpha \geq 4$	$K_x = 4$	$C_x = 1$ for $\lambda \leq 0.83$ $C_x = 1.13\left(\frac{1}{\lambda} - \frac{0.22}{\lambda^2}\right)$ for $\lambda > 0.83$		
			$\alpha < 4$	$K_x = 4 + 2.74\left[\frac{4-\alpha}{3}\right]^4$			
	12 	—	K_y=K_y determined as per case 2		For $\alpha < 2$: $C_y = C_{y2}$ For $\alpha \geq 2$: $C_y = \left(1.06 + \frac{1}{10\alpha}\right) C_{y2}$ Where: C_{y2}: C_y determined as per case 2		
	13 	—	$\alpha \geq 4$	$K_x = 6.97$	$C_x = 1$ for $\lambda \leq 0.83$ $C_x = 1.13\left(\frac{1}{\lambda} - \frac{0.22}{\lambda^2}\right)$ for $\lambda > 0.83$		
			$\alpha < 4$	$K_x = 6.97 + 3.1\left[\frac{4-\alpha}{3}\right]^4$			
	14 	—	$K_y = \frac{6.97}{\alpha^2} + \frac{3.1}{\alpha^2}\left[\frac{4-1/\alpha}{3}\right]^4$		$C_y = 1$ for $\lambda \leq 0.83$ $C_y = 1.13\left(\frac{1}{\lambda} - \frac{0.22}{\lambda^2}\right)$ for $\lambda > 0.83$		
	15 	—	$K_\tau = \sqrt{3}\left[5.34 + \frac{4}{\alpha^2}\right]$		$C_\tau = 1$ for $\lambda \leq 0.84$ $C_\tau = \frac{0.84}{\lambda}$ for $\lambda > 0.84$		

Amended						Remarks
	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 	–	$K_\tau = \sqrt{3}(0.6 + 4/\alpha^2)$	$C_\tau = 1$ 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.					

Amended	Remarks
Table An6 Buckling and Reduction Factor for Curved Plate Panel with $R/t_p \leq 2500$	UR S35, Section5, Table 4

Case	Aspect ratio	Buckling factor *K*	Reduction factor *C*
1 	$\frac{d}{R} \leq 0.5\sqrt{\frac{R}{t_p}}$	$K = 1 + \frac{2}{3}\frac{d^2}{Rt_p}$	For general application: $C_{ax} = 1$ for $\lambda \leq 0.25$ $C_{ax} = 1.233 - 0.933\lambda$ for $0.25 < \lambda \leq 1$ $C_{ax} = {}^{0.3}/_{\lambda^3}$ for $1 < \lambda \leq 1.5$ $C_{ax} = {}^{0.2}/_{\lambda^2}$ for $\lambda > 1.5$ For curved single fields, e.g. bilge plating, which are bounded by plane panels as shown in **Fig. An9**: $C_{ax} = \frac{0.65}{\lambda^2} \leq 1.0$
	$\frac{d}{R} > 0.5\sqrt{\frac{R}{t_p}}$	$K = 0.267\frac{d^2}{Rt_p}\left[3 - \frac{d}{R}\sqrt{\frac{t_p}{R}}\right] \geq 0.4\frac{d^2}{Rt_p}$	
2 	$\frac{d}{R} \leq 1.63\sqrt{\frac{R}{t_p}}$	$K = \frac{d}{\sqrt{Rt_p}} + 3\frac{(Rt_p)^{0.175}}{d^{0.35}}$	For general application: $C_{tg} = 1$ for $\lambda \leq 0.4$ $C_{tg} = 1.274 - 0.686\lambda$ for $0.4 < \lambda \leq 1.2$ $C_{tg} = \frac{0.65}{\lambda^2}$ for $\lambda > 1.2$ For curved single fields, e.g. bilge plating, which are bounded by plane panels as shown in **Fig. An9**: $C_{ax} = \frac{0.8}{\lambda^2} \leq 1.0$
	$\frac{d}{R} > 1.63\sqrt{\frac{R}{t_p}}$	$K = 0.3\frac{d^2}{R^2} + 2.25\left(\frac{R^2}{dt_p}\right)^2$	
3 	$\frac{d}{R} \leq \sqrt{\frac{R}{t_p}}$	$K = \frac{0.6d}{\sqrt{Rt_p}} + \frac{\sqrt{Rt_p}}{d} - 0.3\frac{Rt_p}{d^2}$	As in load case 2.
	$\frac{d}{R} > \sqrt{\frac{R}{t_p}}$	$K = 0.3\frac{d^2}{R^2} + 0.291\left(\frac{R^2}{dt_p}\right)^2$	

Amended	Remarks

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.

UR S35, Section5, 2.2.7

An5.2.2.7 Applied Normal and Shear Stresses to Plate Panels

The normal stress, σ_x and σ_y (*N/mm*^{2}) to be applied for the overall stiffened panel capacity and the plate panel capacity calculations as given in **An5.2.1.1** and **An5.2.2.1** respectively, are to be taken as follows:

- For FE analysis, the reference stresses as defined in **An4.2.4**.
- 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 requirements, at load calculation points of the considered stiffener or the considered elementary plate panel, as defined in **item (a)** and **item (b)** of **An3.1.2.1** 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 **An3.1.2.1**.

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

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

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

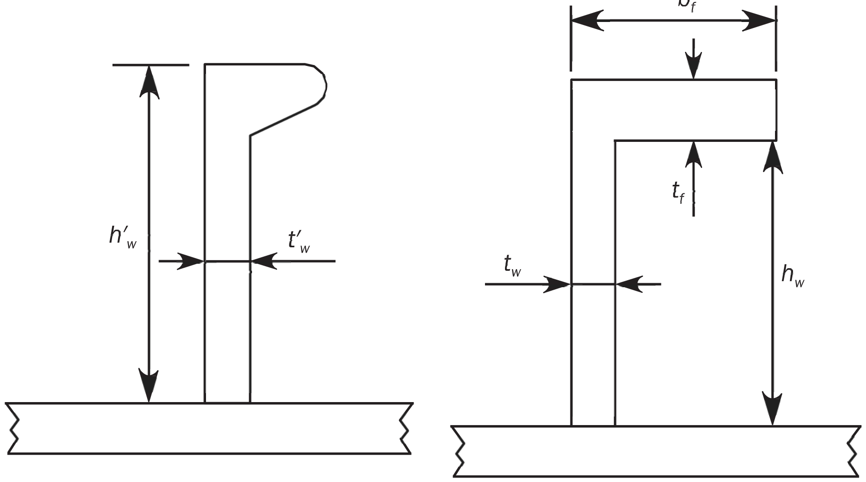
Where:

σ_{xb}, σ_{yb} : Stress (*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 (*N/mm*^{2}) to be applied for the overall stiffened panel capacity and the plate panel capacity calculations as given in **An5.2.1.1** and **An5.2.2.1** respectively, are to be taken as follows:

Amended	Remarks
・For FE analysis, the reference shear stresses as defined in **An4.2.4**. ・For prescriptive assessment of the plate panel capacity, the shear stresses calculated according to the relevant requirements, at load calculation points of the considered elementary plate panel, as defined in **item (a)** of **An3.1.2.1**. ・For prescriptive assessment of the overall stiffened panel capacity, the shear stresses calculated according to the relevant requirements, 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.	
An5.2.3 Stiffeners **An5.2.3.1 Buckling Modes** The following buckling modes are to be checked: ・Stiffener induced failure (*SI*) ・Associated plate induced failure (*PI*)	UR S35, Section5, 2.3.1
An5.2.3.2 Web Thickness of Flat Bar For accounting the decrease of the stiffness due to local lateral deformation, the effective web thickness (*mm*) of flat bar stiffener is to be used in **An5.2.1** and **An5.2.3.4** 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: $t_{w\_red} = t_w \left(1 - \frac{2\pi^2}{3} \left(\frac{h_w}{s} \right)^2 \left(1 - \frac{b_{eff1}}{s} \right) \right)$	UR S35, Section5, 2.3.2

Amended	Remarks
An5.2.3.3 Idealisation of Bulb Profile Bulb profiles are to be considered as equivalent angle profiles. The net dimensions (*mm*) of the equivalent built-up section are to be obtained 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 (*mm*) of a bulb section as shown in **Fig. An10**. α: Coefficient equal to: $\alpha = 1.1 + \frac{(120 - h'_w)^2}{3000}$ for $h'_w \leq 120$ $\alpha = 1.0$ for $h'_w > 120$	UR S35, Section5, 2.3.3

Amended	Remarks
Fig. An10 Idealisation of Bulb Stiffener  Bulb flat Equivalent angle bar	UR S35, Section5, Figure 2
An5.2.3.4 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: $\frac{\gamma_c \sigma_a + \sigma_b + \sigma_w}{\sigma_Y} S = 1$ with the corresponding buckling utilisation factor defined as $\eta_{stiffener} = \frac{1}{\gamma_c}$ Where: σ_a : Effective axial stress (*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 (*N/mm*^{2}) acting on the stiffener with its attached plating.	UR S35, Section5, 2.3.4

Amended-Original Requirements Comparison Table (Buckling Strength Assessments of Ship Structural Elements)

Amended	Remarks
・ For FE analysis, σ_x is the FE corrected stress as defined in **An5.2.3.6** in the attached plating in the direction of the stiffener axis. ・ For prescriptive assessment, σ_x is the axial stress calculated according to **An3.2.2.1** at load calculation point of the stiffener, as defined in **An3.1.2.1**. ・ For grillage beam analysis, σ_x is the stress acting along the *x*-axis of the attached buckling panel. σ_Y: Specified minimum yield stress (N/mm^2) of the material $\sigma_Y = \sigma_{Y\_S}$ for stiffener induced failure (*SI*). $\sigma_Y = \sigma_{Y\_P}$ for plate induced failure (*PI*). σ_b: Bending stress (N/mm^2) in the stiffener $\sigma_b = \frac{M_0 + M_1 + M_2}{1000Z}$ Z: Net section modulus (cm^3) of stiffener including effective width of plating according to **An5.2.3.5**, 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 (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*). M_1: Bending moment (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 (kN/m^2) ・ For FE analysis, P is the average pressure as defined in **An4.2.5.2** in the attached plating. ・ For prescriptive assessment, P is the pressure calculated at load calculation point of the stiffener, as defined in **An3.1.2.1**.	

Amended	Remarks

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 (*Nmm*) due to the lateral deformation *w* of stiffener:

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

γ_{GEB} : Stress multiplier factor of global elastic buckling capacity as defined in **An5.2.1**.

F_E: Ideal elastic buckling force (*N*) of the stiffener

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

I: Moment of inertia (*cm*^{4}) of the stiffener including effective width of attached plating according to **An5.2.3.5**. *I* is to comply with the following requirement:

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

t_p: Net thickness of plate (*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 $\lambda_G > 1.56$

λ_G: The reference degree of global slenderness of the stiffened panel, to be taken as

$$\lambda_G = \sqrt{\frac{\gamma_{\sigma Y}}{\gamma_{GEB}}}$$

Amended	Remarks

$$\gamma_{\sigma Y} = \frac{\min(\sigma_{Y\_P},\sigma_{Y\_S})}{\sqrt{\sigma_{x,av}^2+\sigma_y^2-\sigma_{x,av}\sigma_y+3\tau^2}}$$

$\sigma_{x,av}$: Average stress for both plate and stiffener as defined in **An5.2.1.2**.

σ_y: Applied transverse stress (*N/mm*^{2}) to the plate panel as defined in **An5.2.1.1**.

τ: Applied shear stress (*N/mm*^{2}) to the plate panel as defined in **An5.2.1.1**.

w_0: Assumed imperfection (*mm*) to be taken as:

$$w_0 = \ell/1000$$

σ_w: Stress due to torsional deformation (*N/mm*^{2}) to be taken as:

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

$$\sigma_w = E y_w e_f \Phi_0 \left(\frac{m_{tor}\pi}{l_{tor}}\right)^2 \left(\frac{1}{1-\frac{\gamma\sigma_a}{\sigma_{ET}}}-1\right)$$ 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 (*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 (*N/mm*^{2}) for torsional buckling to be taken as:

Amended	Remarks
$\sigma_{ET} = \frac{E}{I_p}\left[\left(\frac{m_{tor}\pi}{\ell_{tor}}\right)^2 I_\omega \cdot 10^2 + \frac{1}{2(1+\nu)} I_T + \left(\frac{\ell_{tor}}{m_{tor}\pi}\right)^2 \varepsilon \cdot 10^{-4}\right]$ I_p: Net polar moment of inertia (cm^4) of the stiffener about point C as shown in **Fig. An2**, as defined in **Table An7**. I_T: Net St. Venant's moment of inertia (cm^4) of the stiffener as defined in **Table An7**. I_ω: Net sectorial moment of inertia (cm^6) of the stiffener about point C as shown in **Fig. An2**, as defined in **Table An7**. ℓ_{tor}: Stiffener span, distance equal to spacing between primary supporting members, i.e. $\ell_{tor}=\ell$. 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 (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 (mm^2) A_f: Net flange area (mm^2)	

Amended	Remarks
Table An7 Moments of Inertia	UR S35, Section5, Table 5

Table An7 Moments of Inertia

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

(1) t_w is the net web thickness (*mm*). t_{w_red} as defined in **An5.2.3.2** is not to be used in this table.

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

UR S35, Section5, 2.3.5

An5.2.3.5 Effective Width of Attached Plating

The effective width of attached plating of stiffeners b_{eff} (*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,

    $$b_{eff}=\min(\frac{C_{x1}b_1+C_{x2}b_2}{2},\chi_s s)$$

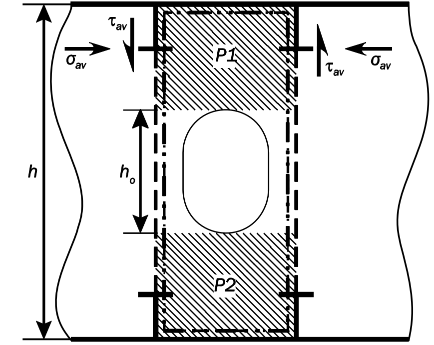
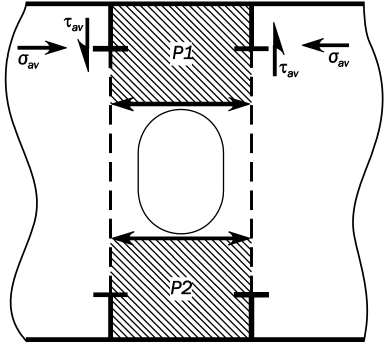
- For $\sigma_x \le 0$:
  - $b_{eff}=\chi_s s$

Amended	Remarks
Where: χ_s: Effective width coefficient to be taken as: $\chi_s = \frac{1.12}{1+\frac{1.75}{\left(\frac{\ell_{eff}}{s}\right)^{1.6}}} \leq 1.0$ for $\frac{\ell_{eff}}{s} \geq 1$ $\chi_s = 0.407\frac{\ell_{eff}}{s}$ for $\frac{\ell_{eff}}{s} < 1$ ℓ_{eff}: Effective length (*mm*) of the stiffener taken as: $\ell_{eff} = \frac{\ell}{\sqrt{3}}$ for stiffener fixed at both ends. $\ell_{eff} = 0.75\ell$ for stiffener simply supported at one end and fixed at the other. $\ell_{eff} = \ell$ for stiffener simply supported at both ends.	
An5.2.3.6 FE Corrected Stresses for Stiffener Capacity When the reference stresses σ_x and σ_y obtained by FE analysis according to **An4.2.4** are both compressive, σ_x is to be corrected according to the following formulae: ・ If $\sigma_x < \nu\sigma_y$ $\sigma_{xcor} = 0$ ・ If $\sigma_x \geq \nu\sigma_y$ $\sigma_{xcor} = \sigma_x - \nu\sigma_y$	UR S35, Section5, 2.3.6
An5.2.4 Primary Supporting Members **An5.2.4.1 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 An8**. The interaction formulae of **An5.2.2.1** are to be used with: ・ $\sigma_x = \sigma_{av}$ ・ $\sigma_y = 0$	UR S35, Section5, 2.4.1

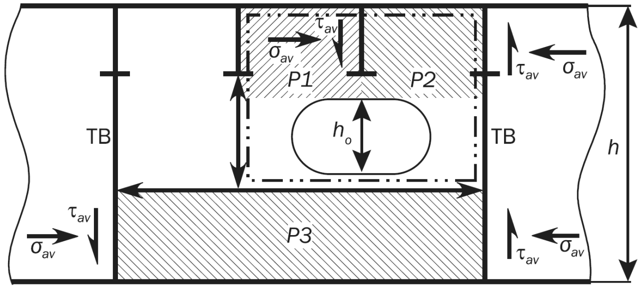
Amended	Remarks
・ $\tau = \tau_{av}$ Where: σ_{av} : Weighted average compressive stress (*N/mm*^{2}) in the area of web plate being considered, i.e. *P1*, *P2*, or *P3* as shown in **Table An8**. For the application of **Table An8**, the weighted average shear stress is to be taken as: ・ Opening modelled in primary supporting members: τ_{av} : Weighted average shear stress (*N/mm*^{2}) in the area of web plate being considered, i.e. *P1*, *P2*, or *P3* as shown in **Table An8**. ・ Opening not modelled in primary supporting members: τ_{av} : Weighted average shear stress (*N/mm*^{2}) given in **Table An8**.	
An5.2.4.2 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 An8**.	UR S35, Section5, 2.4.2

Amended-Original Requirements Comparison Table (Buckling Strength Assessments of Ship Structural Elements)

Amended	Remarks
Table An8 Reduction Factors	UR S35, Section5, Table 6

<table>
<tr><th rowspan="2">Configuration(1)</th><th rowspan="2">C_x, 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 rowspan="2">(a) Without edge reinforcements:(2)
 </td><td rowspan="2">Separate reduction factors are to be applied to areas $P1$ and $P2$ using case 3 or case 6 in Table An5, with edge stress ratio:
$\psi = 1.0$</td><td rowspan="2">Separate reduction factors are to be applied to areas $P1$ and $P2$ using case 18 or case 19 in Table An5</td><td>When case 17 of Table An5 is applicable:
A common reduction factor is to be applied to areas $P1$ and $P2$ using case 17 in Table An5 with:
$\tau_{av} = \tau_{av}\ (web)$</td></tr>
<tr><td>When case 17 of Table An5 is not applicable:
Separate reduction factors are to be applied to areas $P1$ and $P2$ using case 18 or case 19 in Table An5 with:
$\tau_{av} = \tau_{av}\ (web)\ h/(h-h_0)$</td></tr>
<tr><td>(b) With edge reinforcements:
 </td><td>Separate reduction factors are to be applied for areas $P1$ and $P2$ using C_x for case 1 or C_y for case 2 in Table An5 with stress ratio:
$\psi = 1.0$</td><td>Separate reduction factors are to be applied for areas $P1$ and $P2$ using case 15 in Table An5.</td><td>Separate reduction factors are to be applied to areas $P1$ and $P2$ using case 15 in Table An5 with:
$\tau_{av} = \tau_{av}\ (web)\ \ h/(h-h_0)$</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 (a). Panel $P3$ is to be evaluated in accordance with (b).</td></tr>
</table>

<table>
<tr><th>Amended</th><th>Remarks</th></tr>
<tr><td>



Where:

h : Height (m) of the web of the primary supporting member in way of the opening.

h_0: Height (m) of the opening measured in the depth of the web.

τ_{av} *(web)*: Weighted average shear stress (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.

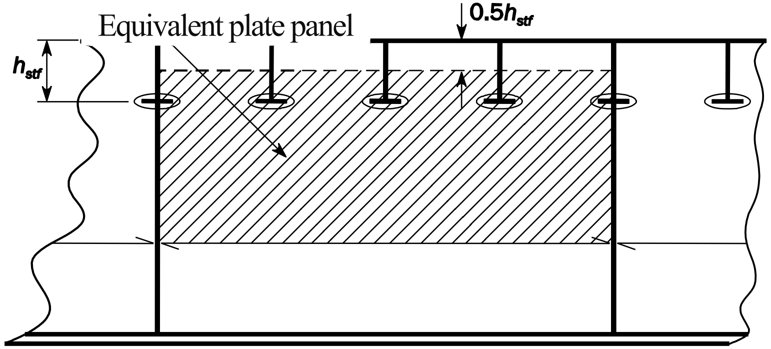
Note (2): For a PSM web panel with opening and without edge reinforcements as shown in configuration (a), the applicable buckling assessment method depends on its specific boundary conditions. If one of the long edges along the face plate or along the attached plating is not subject to “inline support”, 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.

</td><td></td></tr>
<tr><td>

An5.2.4.3

The equivalent plate panel of web plate of primary supporting members crossed by perpendicular stiffeners is to be idealised as shown in **Fig. An11**.

</td><td>UR S35, Section5, 2.4.3</td></tr>
</table>

Amended	Remarks
Fig. An11 Web plate idealization  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.	UR S35, Section5, Figure 3
An5.2.5 Stiffened Panels with U-type Stiffeners **An5.2.5.1 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 **Fig. An3**) 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 An5**, the correction factor F_{long} for U-type stiffeners as defined in **Table An4** is to be used; and in the calculation of K_y as defined in case 2 of **Table An5**, the F_{tran} for U-type stiffeners as defined in **An5.2.2.5** is to be used. ・ The face plate and web plate panels *EPP* b_f and h_w are to be assessed using UP-B model with $F_{long} = 1$ and $F_{tran} = 1$.	UR S35, Section5, 2.5.1
An5.2.5.2 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.	UR S35, Section5, 2.5.2

Amended	Remarks
・ 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.	
An5.3 Buckling Capacity of Column Structures **An5.3.1 Column Buckling of Corrugations** **An5.3.1.1 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 (*N/mm*^{2}) in the member σ_{cr}:Minimum critical buckling stress (*N/mm*^{2}) taken as: $\sigma_{cr} = \sigma_E$ for $\sigma_E \leq 0.5\sigma_{Y\_S}$ $\sigma_{cr} = \left(1 - \frac{\sigma_{Y\_S}}{4\sigma_E}\right)\sigma_{Y\_S}$ for $\sigma_E > 0.5\sigma_{Y\_S}$ σ_E: Elastic column compressive buckling stress (*N/mm*^{2}) according to **An5.3.1.2**. σ_{Y_S}:Specified minimum yield stress (*N/mm*^{2}) of the considered member. For built-up members, the lowest specified minimum yield stress is to be used.	UR S35, Section5, 3.1.1
An5.3.1.2 Elastic Column Buckling Stress The elastic compressive column buckling stress, σ_E (*N/mm*^{2}) of members subject to axial compression is to be taken as: $\sigma_E = \pi^2 E f_{end} \frac{I}{A\ell_{pill}^2} 10^{-4}$ Where: I: Net moment of inertia (*cm*^{4}) about the weakest axis of the cross section A: Net cross-sectional area (*cm*^{2}) of the member ℓ_{pill}: Unsupported length (*m*) of the member	UR S35, Section5, 3.1.2

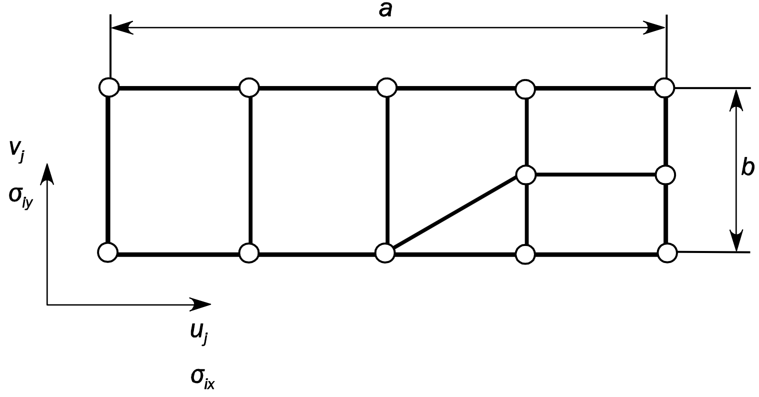
Amended	Remarks
f_{end}: End constraint factor, corresponding to simply supported ends is to be applied except for fixed end support to be used in way of stool with width exceeding 2 times the depth of the corrugation, taken as: ・ $f_{end} = 1.0$ where both ends are simply supported. ・ $f_{end} = 2.0$ where one end is simply supported and the other end is fixed. ・ $f_{end} = 4.0$ where both ends are fixed.	

Amended	Remarks
Appendix 14.6 STRESS BASED REFERENCE STRESSES **Symbols** a: Length (*mm*) of the longer side of the plate panel as defined in **An5** b: Length (*mm*) of the shorter side of the plate panel as defined in **An5** A_i: Area (*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 (*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 (*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 **An5** τ_i: Actual membrane shear stress (*N/mm*^{2}) at the centroid of the i-th plate element of the buckling panel	UR S35, Appendix 1, Symbols
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.	UR S35, Appendix 1, 1.1.1
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 **An4.2.3.1**.	UR S35, Appendix 1, 1.1.2

Amended	Remarks
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.	UR S35, Appendix 1, 1.2.1
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**.	UR S35, Appendix 1, 1.2.2
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 second order polynomial curve as: $\sigma_x(x) = Cx^2 + Dx + E$ 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. $\Pi = \sum_{i=1}^{n} A_i[\sigma_{xi} - (Cx^2 + Dx + E)]^2$ The unknown coefficients C, D and E must yield zero first derivatives, $\partial\Pi$ with respect to C, D and E, respectively.	UR S35, Appendix 1, 2.1.1

Amended	Remarks
$\begin{cases} \frac{\partial \Pi}{\partial C} = 2\sum_{i=1}^{n} A_i x_i^2 [\sigma_{ix} - (Cx_i^2 + Dx_i + E)] = 0 \\ \frac{\partial \Pi}{\partial D} = 2\sum_{i=1}^{n} A_i x_i [\sigma_{ix} - (Cx_i^2 + Dx_i + E)] = 0 \\ \frac{\partial \Pi}{\partial E} = 2\sum_{i=1}^{n} A_i [\sigma_{ix} - (Cx_i^2 + Dx_i + E)] = 0 \end{cases}$ 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: $x_{min} = -\frac{b}{2} - \frac{D}{2C}$ $x_{max} = \frac{b}{2} - \frac{D}{2C}$ 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:	

Amended	Remarks
$\sigma_x = \frac{\sum_{i=1}^{n} A_i \sigma_{xi}}{\sum_{i=1}^{n} A_i}$ 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. 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. $\Pi = \sum_{i=1}^{n} A_i \left[\sigma_{yi} - (A + Bx_i)\right]^2$ The unknown coefficients *A* and *B* must yield zero first partial derivatives, $\partial\Pi$ with respect to *A* and *B*, respectively. $\frac{\partial\Pi}{\partial A} = 2\sum_{i=1}^{n} A_i \left[\sigma_{yi} - (A + Bx_i)\right] = 0$ $\frac{\partial\Pi}{\partial B} = 2\sum_{i=1}^{n} A_i x_i \left[\sigma_{yi} - (A + Bx_i)\right] = 0$ The unknown coefficients *A* and *B* are obtained by solving the 2 above equations and are given as follow: $A = \frac{\left(\sum_{i=1}^{n} A_i \sigma_{yi}\right)\left(\sum_{i=1}^{n} A_i x_i^2\right) - \left(\sum_{i=1}^{n} A_i x_i\right)\left(\sum_{i=1}^{n} A_i x_i \sigma_{yi}\right)}{\left(\sum_{i=1}^{n} A_i\right)\left(\sum_{i=1}^{n} A_i x_i^2\right) - \left(\sum_{i=1}^{n} A_i x_i\right)^2}$ $B = \frac{\left(\sum_{i=1}^{n} A_i\right)\left(\sum_{i=1}^{n} A_i x_i \sigma_{yi}\right) - \left(\sum_{i=1}^{n} A_i x_i\right)\left(\sum_{i=1}^{n} A_i \sigma_{yi}\right)}{\left(\sum_{i=1}^{n} A_i\right)\left(\sum_{i=1}^{n} A_i x_i^2\right) - \left(\sum_{i=1}^{n} A_i x_i\right)^2}$ The transverse stress is to be taken as: $\sigma_y = \max(A, A + Ba)$	UR S35, Appendix 1, 2.1.2

Amended	Remarks
The edge stress ratio is to be taken as: $\psi_y = \frac{min(A,A+Ba)}{max(A,A+Ba)}$ for $\sigma_y > 0$ ψ_y=1 for $\sigma_y \leq 0$	
Fig. 1 Buckling Panel  	UR S35, Appendix 1, Figure 1
2.1.3 Shear Stress The shear stress τ is to be calculated using a weighted average approach and is to be taken as: $\tau = \frac{\sum_{i=1}^{n} A_i \tau_i}{\sum_{i=1}^{n} A_i}$	UR S35, Appendix 1, 2.1.3

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

$$\psi_x = 1$$

$$\psi_y = 1$$

UR S35, Appendix 1, 2.2.1

EFFECTIVE DATE AND APPLICATION

1. The effective date of the amendments is 1 July 2024.
2. Notwithstanding the amendments to the Rules, the current requirements apply to ships for which the date of contract for construction* is before the effective date.
3. For ships subject to Part C of the Rules for the Survey and Construction of Steel Ships and prior to its comprehensive revision by Rule No.62 on 1 July 2022, and which the date of contract for construction* is on and after the effective date, this amendment is applied.

* "contract for construction" is defined in the latest version of IACS Procedural Requirement (PR) No.29.

IACS PR No.29 (Rev.0, July 2009)

1. The date of "contract for construction" of a vessel is the date on which the contract to build the vessel is signed between the prospective owner and the shipbuilder. This date and the construction numbers (i.e. hull numbers) of all the vessels included in the contract are to be declared to the classification society by the party applying for the assignment of class to a newbuilding.
2. The date of "contract for construction" of a series of vessels, including specified optional vessels for which the option is ultimately exercised, is the date on which the contract to build the series is signed between the prospective owner and the shipbuilder.

Amended-Original Requirements Comparison Table (Buckling Strength Assessments of Ship Structural Elements)

Amended	Remarks
For the purpose of this Procedural Requirement, vessels built under a single contract for construction are considered a "series of vessels" if they are built to the same approved plans for classification purposes. However, vessels within a series may have design alterations from the original design provided: (1) such alterations do not affect matters related to classification, or (2) If the alterations are subject to classification requirements, these alterations are to comply with the classification requirements in effect on the date on which the alterations are contracted between the prospective owner and the shipbuilder or, in the absence of the alteration contract, comply with the classification requirements in effect on the date on which the alterations are submitted to the Society for approval. The optional vessels will be considered part of the same series of vessels if the option is exercised not later than 1 year after the contract to build the series was signed. **3.** If a contract for construction is later amended to include additional vessels or additional options, the date of "contract for construction" for such vessels is the date on which the amendment to the contract, is signed between the prospective owner and the shipbuilder. The amendment to the contract is to be considered as a "new contract" to which **1.** and **2.** above apply. **4.** If a contract for construction is amended to change the ship type, the date of "contract for construction" of this modified vessel, or vessels, is the date on which revised contract or new contract is signed between the Owner, or Owners, and the shipbuilder. Note: This Procedural Requirement applies from 1 July 2009.	