DNV·GL

RULES FOR CLASSIFICATION

Yachts

Edition October 2016
Amended January 2018

Part 3 Hull

Chapter 4 Metallic hull girder strength and local scantlings

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DNV GL AS

FOREWORD

DNV GL rules for classification contain procedural and technical requirements related to obtaining and retaining a class certificate. The rules represent all requirements adopted by the Society as basis for classification.

Any comments may be sent by e-mail to *rules@dnvgl.com*

CURRENT – CHANGES

This document supersedes the October 2016 edition DNVGL-RU-YACHT Pt.3 Ch.4.

Changes in this document are highlighted in red colour. However, if the changes involve a whole chapter, section or sub-section, normally only the title will be in red colour.

Amendments January 2018

— Correction of references have been made
— Sec.3 [7.3] : Formula adapted to previous chapters.

Amendments July 2017

Editorial changes have been made.

Main changes October 2016, entering into force as from date of publication

• General

— Completely revised to provide compatibility for motor and sailing yachts
— PSF scantling concept has been waived and a conventional concept introduced
— Re-structured for improved clarity
— Sec.3, Sec.4 and App.A have updated titles.

• Sec.1 Introduction

— Applicability adjusted to Ch.3 *Hull design loads*.

• Sec.2 Hull girder strength

— Re-structured for improved logic.

• Sec.3 Hull local scantlings

— Re-structured for improved logic
— Applicability adjusted to Ch.3 by waiving PSF concept and introducing local stress coefficients
— Applicability adjusted to Ch.3 *Hull design loads*
— C_a factor (panel aspect ratio) has been adapted to plate theory of Roark and Young
— Corrosion allowance clarified
— Pillar dimensioning expanded by superimposed bending.

• App.D Fatigue

— Merged with similar App.A from Ch.7.

Editorial corrections

In addition to the above stated changes, editorial corrections may have been made.

CONTENTS

SECTION 1 INTRODUCTION

1 General

1.1 Applicability

This chapter provides the scantling design criteria for the following craft built from metallic materials:

— sailing yachts
— motor yachts
— high speed yachts
— multihulls, as per definitions given in Pt.5.

It is also applicable to parts constructed in metallic materials on vessels built in composites. The criteria defined in this chapter for hull girder strength and local scantlings shall be applied using the design loads derived from Ch.3 Sec.2 and/or Ch.3 Sec.3, under consideration of App.A to App.E.

SECTION 2 HULL GIRDER STRENGTH

For symbols not defined in this section, see Ch.1 Sec.4.

1 General

1.1 Scope

This section specifies the criteria for calculating the hull girder strength characteristics, in association with the hull girder loads specified in Ch.3 Sec.3.

Sec.3 specifies the hull girder design stresses to be used for the evaluation of the scantling of structural members according to Sec.4 subject to local load and hull girder load combination.

1.2 Introduction

Global loads are considered loads acting on the hull without the consideration of local load introduction.

The expression hull girder implies that the hull structure will be modelled as a single beam. This will be followed by a generic section analysis which includes the review of the global strength capacity of 2-dimensional beam cross sections.

This is provided the structure allows for such simplification.

For structures with a high complexity exceeding the applicability of what is described above, an appropriate 3-dimensional analysis will be required.

Acceptable methods and procedures of analysing the structures in 3D are subject to agreement with the Society.

2 Cross sections

2.1 General

2.1.1 Hull girder cross transverse sections shall be considered as being constituted by the members contributing to the hull girder longitudinal strength, i.e. all continuous longitudinal members below and including the strength deck defined in Ch.1, taking into account the requirements in [2.1.2] to [2.1.10].

For the calculation of the required section moduli at each cross section along the yacht length *L* the most unfavourable value of the total vertical bending moments shall be taken into account. These moments shall be determined by the following combinations:

$M_{sw,max} + M_{wv,h}$ for the maximum vertical bending moment, or

$M_{sw,min} + M_{wv,s}$ for the minimum vertical bending moment.

2.1.2 Structural members not contributing to hull girder sectional area

The following members shall not be considered in the calculation as they are considered not contributing to the hull girder sectional area:

— superstructures above strength deck
— inner (accommodation) walls
— vertically corrugated bulkheads
— bulwarks
— fender profiles
— bilge keels
— sniped or non-continuous longitudinal stiffeners.

2.1.3 Longitudinal bulkheads with vertical corrugations

For longitudinal bulkheads with vertical corrugations, the vertical corrugations shall not be included in the hull girder transverse section. Longitudinal bulkheads with vertical corrugations are not effective for hull girder bending, but they are effective for hull girder shear force.

2.1.4 Members in various materials

Where members contributing to the longitudinal strength are made in various materials, the equivalent sectional area included in hull girder transverse section can be calculated by considering the ratio of the Young's modulus.

2.1.5 Definitions of openings

The following definitions of opening shall be applied:

a) Large openings are i.e. openings exceeding 2.5 m in length or 1.2 m in breadth.
b) Small openings are i.e. manholes, lightening holes, etc. that are not large ones.
c) Isolated openings are openings spaced not in the area of deck stringer plate and shearstrake.

2.1.6 Large openings

Large and non-isolated openings shall be deducted from the sectional area used in hull girder moment of inertia and section modulus.

2.1.7 Isolated small openings

A deduction-free sum of isolated small opening breadths Σb_S in one transverse section in the strength deck or bottom area may be considered equivalent to a reduction in section modulus at deck or bottom by 3% if

$$\Sigma b_S \le 0.06(B - \Sigma b)$$

Σb_S = total breadth of small openings, in m, in the strength deck or bottom area at the transverse section being considered, determined as indicated in Figure 1

Σb = total breadth of large openings, in m, at the transverse section being considered, determined as indicated in Figure 1.

Where the total breadth of small openings Σb_S does not fulfil the above criteria, only the excess of breadth shall be deducted from the sectional areas included in the hull girder transverse sections.

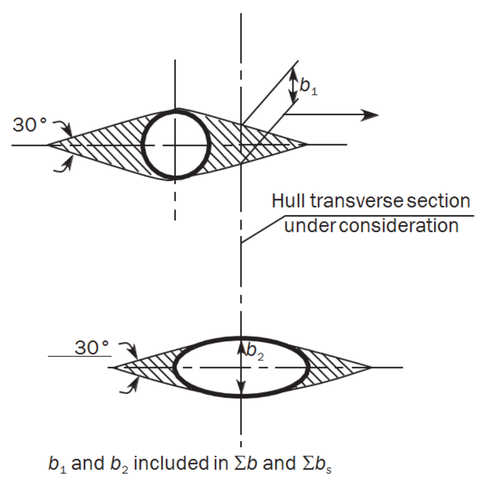


Figure 1 Calculations of Σb and Σb_s

2.1.8 Lightening holes, draining holes and single scallops

Lightening holes, draining holes and single scallops in longitudinals need not be deducted if their height is less than 0.25% the web height of the longitudinals, in mm. Otherwise, the excess shall be deducted from the sectional area or compensated.

2.1.9 Non-continuous decks and longitudinal bulkheads

When calculating the effective area in way of non-continuous decks and longitudinal bulkheads, the effective area shall be taken as shown in Figure 2 and Figure 3. The shadow area, which indicates the ineffective area, is obtained by drawing two tangent lines with an angle of 15 degree to the longitudinal axis of the ship.

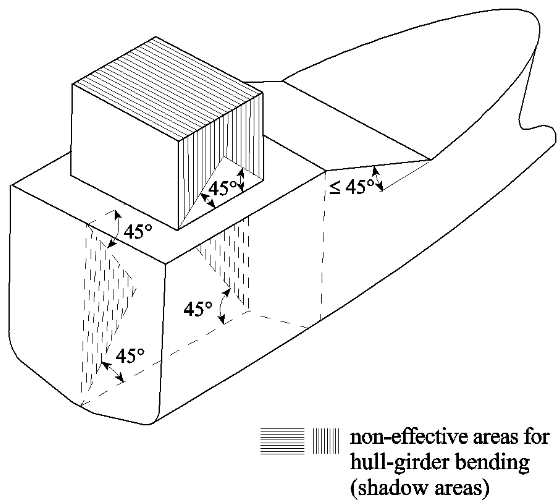


Figure 2 Application of the shadow principle for the ship's hull and superstructures

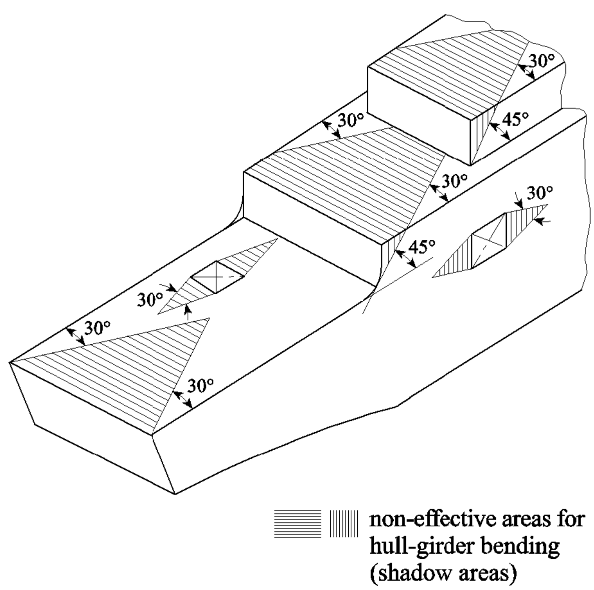


Figure 3 Application of the shadow principle for the ship's hull and superstructures

2.1.10 Longitudinal members with reduced effectiveness

For hull girder cross sections where the beam theory is not applicable, the contribution of longitudinal members with reduced effectiveness (e.g. superstructure above strength deck) to the strength characteristics of the hull girder shall be determined by direct calculations.

2.2 Calculation of section modulus

The vertical section modulus Z in m^3, at any point of a hull transverse section is obtained from the following equation:

$$Z = I_Y / |z - z_n|$$

The moments of inertia, I_Y in m^4, are those, calculated about the horizontal neutral axes of the offered section being considered.

3 Design verifications

3.1 Permissible stress method

Along the length L the equivalent stress σ_v in MPa, at any point considered shall comply with the following formula:

$$\sigma_v = \sqrt{\sigma^2 + 3 \cdot \tau^2} \leq f_m \cdot R_y$$

3.2 Proof of buckling strength

In case of structural members contributing to the longitudinal strength and subjected to compressive stresses resulting from the total vertical bending moment and/or subjected to shear stresses resulting from the total vertical shear force shall be examined for sufficient resistance to buckling according to App.B.

The stresses shall be determined according to Sec.3 [3.1.1].

3.3 Ultimate capacity method

Alternatively to [3.1] and [3.2] the dimensioning of longitudinal structures may be verified by providing the ultimate capacity according to [3.3.1] and [3.3.2]. The calculations shall include those structural elements contributing to the hull girder longitudinal strength and are based on offered scantlings.

The vertical hull girder ultimate bending capacity shall be derived for hogging and sagging conditions.

The hull girder ultimate strength requirement given applies along the full length of the hull girder.

This section specifies the criteria for calculating the hull girder strength characteristics to be used for the checks, in association with the hull girder loads specified in Ch.3.

Following conditions shall be considered for wave induced design loads:

— intact condition
— damaged condition, if applicable, according to Ch.10.

Ultimate vertical bending criterion:

$$\left|M_{SW} + M_{WV}\right| \leq \left|\frac{M_U}{\gamma_m}\right|$$

$$M_{SWf} + M_{WVf}\Big| \leq \left|\frac{M_U}{\gamma_m}\right|$$

Ultimate vertical shear criterion:

$$|Q_{SW} + Q_{WV}| \leq \left|\frac{Q_U}{\gamma_m}\right|$$

$$|Q_{SWf} + Q_{WVf}| \leq \left|\frac{M_U}{\gamma_m}\right|$$

where:

γ_m = 1.05

3.3.1 Hull girder ultimate shear capacity

The ultimate shear capacities, in MN, of a hull girder transverse section, in hogging and sagging conditions, are defined as:

$$Q_U = \frac{10^{-6}}{\sqrt{3}} \cdot \sum_{i=1}^{q} \kappa_{\tau i} \cdot b_i \cdot t_i \cdot R_{\gamma,i}$$

q = number of shear force transmitting plate fields (in general, these are only the vertical plate fields of the ship's transverse section, e.g. shell and longitudinal bulkhead plate fields)

$\kappa_{\tau i}$ = reduction factor of the i^{th} plate field according to App.B [2.1]

b_i = breadth of the i^{th} plate field [mm]

t_i = thickness of the i^{th} plate field [mm]

R_{Yi} = minimum nominal yield point of the i^{th} plate field, in MPa.

3.3.2 Hull girder ultimate bending capacity

The ultimate bending moment capacities of a hull girder transverse section, in hogging and sagging conditions, are defined as the maximum values of the curve of bending moment capacity versus the curvature χ of the transverse section considered (see Figure 4). The curvature χ is positive for hogging condition and negative for sagging condition.

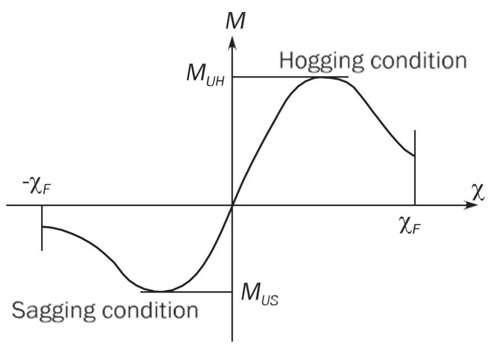


Figure 4 Bending moment capacity versus curvature χ

The hull girder ultimate bending capacity, M_U, shall be assessed by prescriptive method given in App.E or by non-linear FE.

3.4 Fatigue assessment

The required welding details and classifying of notches result from the fatigue strength analysis according to App.D. For this purpose the probability factor f_Q may be reduced according to Ch.3 Sec.3 corresponding to a probability level of $Q = 10^{-4}$.

SECTION 3 HULL LOCAL SCANTLINGS

For symbols not defined in this section, see Ch.1 Sec.4.

ψ = 1.0 in general or for major load of load combination, unless otherwise specified

f_m = $\frac{R_m}{1.5 \cdot R_y}$

= < 1.0 (for isotropic plate material)

γ = local stress coefficient as given in Table 1

f_p = < 1.5

Guidance note:

For the purpose of determining the local stress coefficient γ for watertight bulkheads, following values shall be used for f_p:

= 1.0 for sandwich panels

= 1.10 for corrugated bulkhead elements

= 1.5 for plates and flat bars with plating

= 1.4 for bulb profiles with plating

= 1.25 for rolled angle bars with plating

= 1.15 for T-profiles with plating.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

1 Definitions

The local stress coefficients for determining the scantlings of structural elements are defined in Table 1.

Table 1 Local stress coefficients γ

Structural applications	γ
Hull, weather exposed structures	1.4
WT bulkhead	1.1/ f_p
Tank boundaries for p_{T1}	1.4
Tank boundaries for p_{T2}	1.2
Chain locker, pools	1.2
Deck (accommodation, mooring, machinery, cargo)	1.4
Deck (heli normal landing)	1.0
Deck (heli crash, emergency landing)	1.2/f_p
Bilge keel	1.0
Sea chest	1.4
Bulbous bow, plate stem	1.0
Docking	1.0

2 General

2.1 Application

2.1.1 This section applies to hull structures over the full length of the ship.

2.1.2 This section provides requirements for evaluation of plating, stiffeners and primary supporting members (PSM) subject to lateral pressure, local loads and hull girder loads, as applicable.

2.1.3 Requirements are specified for:

— load application in [3]
— minimum thickness of plates, brackets, stiffeners and PSM in [4]
— plating in [5]
— stiffeners and PSM in [6]
— pillars in [7].

In addition, other requirements not related to defined design load sets, are provided.

2.1.4 Additional local strength requirements for various structural hull members are specified for:

— bottom and shell structures in Sec.4 [1]
— tank structures in Sec.4 [2]
— decks and walls in Sec.4 [3]
— watertight and non-watertight bulkheads in Sec.4 [4].

2.1.5 Required scantlings
The offered scantling shall be greater than or equal to the required scantlings based on requirements provided in this section.

3 Load application

3.1 Load components

3.1.1 Hull girder bending
This paragraph specifies the hull girder stresses to be superimposed with those of the scantlings of structural members subject to local loads.

For seagoing condition the longitudinal normal stress, σ_L and shear stress τ_L, in MPa, induced by acting vertical (and horizontal if applicable) bending moments and shear forces are obtained from the following equations:

$$\sigma_L = \sigma_{SW} + f_Q \cdot \sqrt{\sigma_{WV}^2 + \sigma_{WH}^2}$$

$$\tau_L = |\tau_{SW}| + f_Q|\tau_{WV}|$$

$$\sigma_L = \max(\sigma_{Li})\,;\tau_L = \max(\tau_{Li})$$

Longitudinal normal stress induced by:

static hull girder bending σ_{SW}	$= M_{SW} \cdot \frac{z - z_n}{I_y}$
dynamic vertical hull girder bending σ_{WV}	$= M_{WV} \cdot \frac{z - z_n}{I_y}$
dynamic horizontal hull girder bending σ_{WH}	$= M_{WH} \cdot \frac{y}{I_z}$

Shear stress induced by:

static hull girder shear force τ_{SW}	$= 1000 \cdot Q_{SW} \cdot \frac{S_y(z)}{I_y \cdot t}$
dynamic hull girder shear force τ_{WV}	$= 1000 \cdot Q_{WV} \cdot \frac{S_y(z)}{I_y \cdot t}$

where:

$S_y(z)$ = first moment of the sectional area considered m^3 above or below, respectively, the level z considered, and related to the neutral axis

t = thickness of side shell plating considered [mm].

In general stresses due to torsion can be neglected for yachts with closed weather decks or weather decks with small openings. For yachts with large deck openings and/or unusual structural design stresses due to torsion shall be considered in a global stress analysis.

The stress components (with the proper signs) shall be added such that for σ_L and τ_L extreme values are resulting.

f_Q = probability factor according to Ch.3 Sec.3 for $Q = 10^{-6}$.

Shear stress distribution shall be calculated by calculation procedures approved by the Society. For ships with multi-cell transverse cross sections, the use of such a calculation procedure, especially with non-uniform distribution of the load over the ship's transverse section, may be stipulated.

Guidance note:

As a first approximation σ_L and τ_L may be taken as follows for

— bottom plating:

$$\sigma_L = 0.5 \cdot R_y$$

$$\tau_L = 0$$

— side shell plating:

$$\sigma_L = 0.4 \cdot R_y$$

$$\tau_L = 0.25 \cdot R_y$$

— deck plating:

$$\sigma_L = 0.70 \cdot R_y$$

$$\tau_L = 0$$

with f_m according to Sec.4.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

3.1.2 Lateral pressures

The static and dynamic lateral pressures in intact condition induced by the sea and the various types of ballast and other liquids shall be considered. Applied loads will depend on the location of the elements under consideration, and the adjacent type of compartments.

Watertight boundaries of compartments not intended to carry liquids, excluding hull envelope, shall be subjected to lateral pressure in flooded conditions.

3.1.3 Pressure combination

For elements of the outer shell following load combination shall be taken into account, if the compartment adjacent to the outer shell is:

— intended to carry liquids:

  the lateral pressures to be considered are not less than the maximum differences between the internal pressures and the external sea pressures at the corresponding draught

— not intended to carry liquids:

  the internal pressures and external sea pressures shall be considered independently.

For elements other than those of the outer shell, the lateral pressures on an element separating two adjacent compartments are those obtained considering the two compartments individually loaded.

4 Minimum thicknesses

4.1 Plating

The thickness of plating in mm, shall not be less than the minimum thickness t_{min} requirement given in Table 2.

Table 2 Minimum thickness of plating

Elements of the hull structure		Minimum thickness t_{min} [mm]
Designation	*Reference position*	
Flat plate keel	Sec.5	$14 \cdot \sqrt{\frac{L}{R_y}}$
Shell plating: $z \le T + c_0/2$ [1)]		$10 \cdot \sqrt{\frac{L}{R_y}}$
$z > T + c_0/2$		$7 \cdot \sqrt{\frac{L}{R_y}}$
Chain locker	Ch.7 Sec.4 [5]	5.0
All strength relevant structural openings	Various sections	3.0
1) For c_0 see Ch.2 Sec.2 [1.2]		

4.2 Stiffeners and tripping brackets

The minimum thickness of the web and face plate, if any, of stiffeners and tripping brackets in mm, shall not be less than 3.0 mm.

In addition, the offered thickness of the web of stiffeners and tripping brackets, in mm, shall be:

— not less than 40% of the required thickness *t* of the attached plating, to be determined according to [5]
— less than twice the offered thickness of the attached plating.

4.3 Primary supporting members

The minimum thickness of the web plating and flange of primary supporting members in mm, shall not be less than 3.0 mm.

5 Plating

For symbols not defined in this section, refer to Ch.1 Sec.4.

a = breadth of smaller side of the plate panel [m]
b = breadth of larger side of the plate panel [m]
c = single point load distribution factor
r = radius of curvature [m]
α = aspect ratio of single plate field
= $\frac{a}{b} \le 1.0$
β = $\frac{1}{\alpha}$
c_a = factor considering the aspect ratio of the plate panel

$$= \frac{1}{\sqrt{1+\left(\frac{2.5-\beta}{\beta^2+0.9}\right)^2}} \text{ for } \beta < 2.5$$

$= 1.0$ for $\beta \geq 2.5$

c_r = factor for curved plates

$= 1 - \frac{a}{2 \cdot r}$ for curved plates

$= \geq 0.75$

= 1 for flat plates

σ_L = hull girder bending stress, see Sec.2 [2.2]

τ_L = shear stress due to hull girder bending, see Sec.2 [2.2]

p = lateral design pressure in kPa, at load point for the defined load case, each

F_E = wheel or single point load in kN

γ_m = 1.1

f_L = stress coefficient due to super imposed hull girder stress shall not be taken greater than 0.77 for shell plating otherwise not greater than 1.0:

$$= 1.125 \cdot \left[\sqrt{1.0 - 3 \cdot \left(\frac{\tau_L}{R_y}\right)^2 - 0.786 \cdot \left(\frac{\sigma_L}{R_y}\right)^2} - 0.062\frac{\sigma_L}{R_y}\right]$$

For longitudinally stiffened plates (larger side of plate parallel to ships longitudinal direction, parallel to direction of σ_L)

$$= 1.125 \cdot \left[\sqrt{1.0 - 3 \cdot \left(\frac{\tau_L}{R_y}\right)^2} - 0.89\frac{\sigma_L}{R_y}\right]$$

For transversely stiffened plates (larger side of plate transverse to ships longitudinal direction, transverse to the direction of σ_L).

5.1 Corrosion additions

5.1.1 General

The corrosion additions given in these rules are applicable to carbon-manganese steels, stainless steels and aluminium alloys.

The considered corrosion addition shall not be less than the respective thickness taken as 0.5 mm for these materials.

The required thickness of the plating, t in mm, shall not be less than:

$$t = t' + t_c$$

5.1.2 Steel

Based on the calculated values the scantling determination requires the corrosion addition t_c to the nominal plate thickness:

— t_c = 0.5 mm in general
— t_c = 0.7 mm for lubrication oil tanks, gas oil tanks or equivalent tanks
— t_c = 1.0 mm for ballast water tanks, sewage tanks, sea chests
— t_c = 1.0 mm for deck plating below elastically mounted deckhouses
— t_c = 2.0 mm for chain locker, stern tube

— for special applications t_c shall be agreed with the Society.

For stainless steel a corrosion addition is not required. A reserve thickness of 0.5 mm is considered within t_c.

5.1.3 Aluminium alloy

If the measures for corrosion protection as described in Pt.2 and App.A [6.3] are fully applied, a corrosion addition is not required for the aluminium alloys defined in App.A. A reserve thickness of 0.5 mm is considered within t_c.

In any case, any direct contact between steel and aluminium alloy (e.g. aluminium chain locker and chain cable) shall be avoided.

5.2 Design criteria for plating

The offered plate thickness, t_{off} in mm, shall not be taken less than the required thickness t or t_{min}, respectively (the greater value shall be taken) considering following rounding-off tolerances:

— Where in determining plate thicknesses in accordance with the provisions of the following sections the figures differ from full or half mm, they may be rounded off to full or half millimetres up to 0.2 or 0.7; above 0.2 and 0.7 mm they shall be rounded up.

— If the under-thickness tolerance for extrusions is more than 7%, the exceeding difference shall be considered for the calculations.

5.3 Plating subject to lateral pressure

The nominal thickness, t' in mm, shall not be taken less than:

$$t' = 15.81 \cdot a \cdot \sqrt{\frac{p \cdot \gamma \cdot \gamma_m}{R_y \cdot f_m \cdot f_L}} \cdot c_a \cdot c_r$$

Note:

In cases in which a hull is faired with an additional fairing compound, it should be made sure that the occurring strain in the fairing compound does not exceed its pertinent limits.

---e-n-d---o-f---n-o-t-e---

5.4 Plating subject to lateral wheel and single point loads

The nominal thickness, t' in mm, of plating for single point or wheel loads shall not be taken less than the greatest value for all applicable design load sets given by:

$$t' = c \cdot \sqrt{\frac{F_E \cdot \gamma_m \cdot \gamma}{R_y \cdot f_m}}$$

5.4.1 Wheel loads

The load distribution factor c for a wheel or a group of wheels shall be determined according to the following equations:

for the aspect ratio $\beta = 1$:

for the range $0 < \frac{f}{F} < 0.3$

$c = 26 - \sqrt{\frac{f}{F} \cdot \left(656 - 849 \cdot \frac{f}{F}\right)}$

for the range $0.3 \leq \frac{f}{F} \leq 1,0$:

for the range $c = 16.7 - 5.60 \cdot \frac{f}{F}$:

$0 < \frac{f}{F} < 0.3$

for the aspect ratio β ≥ 2.5:

for the range $0 < \frac{f}{F} < 0.3$

$c = 27.8 - \sqrt{\frac{f}{F} \cdot \left(1003 - 1389 \cdot \frac{f}{F}\right)}$

for the range $0.3 \leq \frac{f}{F} \leq 1,0$:

$c = 16.7 - 7.2 \cdot \frac{f}{F}$

for intermediate values of β the factor c shall be obtained by linear interpolation.

f = print area of wheel or group of wheels in m^2. If it is not known the definition of Ch.3 Sec.3 [3.4.3] can be used

F = area of plate panel $\beta\, a^2$ in m^2, but need not be taken greater than 2.5 a^2.

5.4.2 Single point loads

For forces e.g. induced by a tug or fender into the shell plating following load distribution factor c can be taken:

c = 10 in general.

6 Stiffeners and primary supporting members

$p_{A,B}$ = lateral design pressure in kPa, at end points A and B, respectively
N = sum of local and global forces in kN, in longitudinal direction of the structural element
N_x = local axial force, in kN
F_s = nominal shear force, in kN according to Sec.3 Table 3
M_b = nominal bending moment, in kNm according to Sec.3 Table 3
ℓ = unsupported span, in m, according to Sec.3 Table 3 and App.A Figure 3
a = spacing, in m of the secondary stiffeners
e = load width, in m for primary members
= spacing of primary members (in general)
= unsupported length of secondary stiffeners
m_a = factor considering aspect ratio of stiffeners
$= \frac{a}{6 \cdot \ell} \cdot \left[4 - \left(\frac{a}{\ell}\right)^2\right]$

a need not be greater than ℓ

m_n = factor considering aspect ratio of girders
= 0

n_a = number of supported secondary stiffeners within the length ℓ

j = number of constraint (fixed) end conditions
= 2 for both ends fixed
= 1.0 for one end simply supported and one end fixed
= 0 for both ends simply supported

ℓ_{ki} = length of end connection in m, see Sec.3 Table 3

α_i = angle of end connection, in degree, see Sec.3 Table 3

n_k = factor considering end connection

$$= \left[1 - \frac{\Sigma\, \ell_{ki} \cdot \sin^2 \alpha_i}{\ell}\right]^2$$

W_{el} = elastic section modulus in cm^3, including effective width of plating according to [4] - Minimum thicknesses

W_{pl} = plastic section modulus in cm^3, including effective width of plating according to [4] - Minimum thicknesses
$\leq f_p\, W_{el}$ for the purpose of determining the bending capacity

6.1 Definitions

6.1.1 Nominal loads for stiffeners and primary members

Nominal shear forces and bending moments for both, elastic and ultimate strength assessment, can be derived from the equations in Table 3 for trapezoidal loaded beams with constant cross sections and typical end connections.

Following definitions shall be taken into account:

— for stiffeners:

$s = a$

$m = m_a$

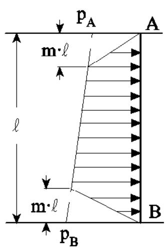
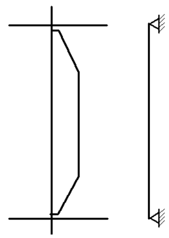
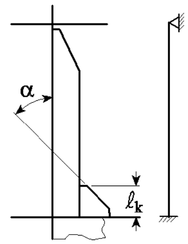
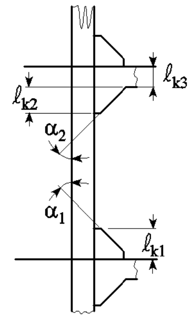
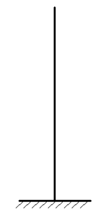
— for PSM:

$s = e$

$m = m_n$.

For other types of load combinations e.g. concentrated loads and/or other end connections the nominal shear forces and bending moments have to be determined by direct calculation.

Table 3 Determination of elastic stresses and ultimate nominal loads for beams

Type of the end connection				
Arrangement and loads	*Simple support at both ends*	*Simple support at one end, constraint at other end*	*Constraint at both ends*	*Cantilever*
				
Calculation of details for elastic strength assessment:				
Elastic shear force [kN]	$F_{A0} = \frac{s \cdot \ell}{6} \cdot \{p_A[2 - m(3 - m)] + p_B(1 - m^2)\}$ $F_{B0} = \frac{s \cdot \ell}{6}\{p_A(1 - m^2) + p_B[2 - m(3 - m)]\}$	$F_A = F_{A0} + \frac{M_B}{\ell}$ $F_B = F_{B0} + \frac{M_B}{\ell}$	$F_A = F_{A0} + \frac{M_A - M_B}{\ell}$ $F_B = F_{B0} + \frac{M_A - M_B}{\ell}$	$F_A = 0$ $F_B = \frac{s \cdot \ell}{2} \cdot (p_A + p_B)$

Type of the end connection				
Arrangement and loads	*Simple support at both ends*	*Simple support at one end, constraint at other end*	*Constraint at both ends*	*Cantilever*
Elastic bending moment [kNm]	$M_0 = \frac{s \cdot \ell^2}{12} \cdot (0.75 - m^2) \cdot (p_A + p_B)$	$M_B = \frac{s \cdot \ell^2}{12} \cdot n_k \cdot [p_A(0.7 - 0.925m^2) + p_B(0.8 - 0.925m^2)]$	$M_A = \frac{s \cdot \ell^2}{12} \cdot n_k \cdot [p_A(0.6 - 1.15m^2) + p_B(0.4 - 0.35m^2)]$ $M_B = \frac{s \cdot \ell^2}{12} \cdot n_k \cdot [p_A(0.4 - 0.35m^2) + p_B(0.6 - 1.15m^2)]$	$M_A = 0$ $M_B = \frac{n_k \cdot s \cdot \ell^2}{6} \cdot (2 \cdot p_A + p_B)$
Elastic design shear stress [MPa]		$\tau_B = \frac{F_B \cdot 10}{A_{SB}}$	$\tau_A = \frac{F_A \cdot 10}{A_{SA}}$ $\tau_B = \frac{F_B \cdot 10}{A_{SB}}$	$\tau_B = \frac{F_B \cdot 10}{A_{SB}}$
Elastic design bending stress [MPa]	$\sigma = \frac{M_0 \cdot k_{sp} \cdot 10^3}{W_{el}}$	$\sigma_B = \frac{M_B \cdot k_{sp} \cdot 10^3}{W_{el}}$	$\sigma_A = \frac{M_A \cdot k_{sp} \cdot 10^3}{W_{el}}$ $\sigma_B = \frac{M_B \cdot k_{sp} \cdot 10^3}{W_{el}}$	$\sigma_B = \frac{M_B \cdot k_{sp} \cdot 10^3}{W_{el}}$
Other types of load combinations and/or connections are subject to direct calculation.				
Ultimate shear force [kN]	$j = 0$	$j = 1$	$j = 2$	—
	$F_A = F_{A0}$ $F_B = F_{B0}$	$F_A = F_{A0}$ $F_B = F_{B0}$	$F_A = F_{A0}$ $F_B = F_{B0}$	$F_A = 0$ $F_B = \frac{s \cdot \ell}{2} \cdot (p_A + p_B)$
Ultimate bending moment [kNm]	$M = M_0$	$M = \frac{2}{3} \cdot n_k \cdot M_0$	$M = 0.5 \cdot n_k \cdot M_0$	$M_B = \frac{n_k \cdot s \cdot \ell^2}{6} \cdot (2 \cdot p_A + p_B)$

6.2 Scantling verification

6.2.1 General

The nominal elastic shear forces and bending moments shall be taken into account.

6.2.2 Simplified dimensioning method

The following analysis may be used for stiffeners and primary supporting members with constant cross sections (e.g. no cut-outs) within the unsupported length, considering the nominal design loads for bending and shear.

The required web area A_{shr} [cm^2] used as effective shear area and the elastic section modulus W_{el} [cm^3] can be determined as follows:

$$A_{shr} \geq \frac{F_S\sqrt{300} \cdot \gamma \cdot \gamma_m}{R_y \cdot f_m} \text{ [cm}^2\text{]}$$

$$W_{el} \geq \frac{M_b \cdot 10^3 \cdot \gamma \cdot \gamma_m \cdot k_{sp}}{R_y \cdot f_m \cdot f_L} \text{ [cm}^3\text{]}$$

where:

f_L	=	stress coefficient due to superimposed hull girder stress
	=	$1.0 - 0.89 \cdot \frac{\sigma_L}{R_y}$
σ_L	=	normal stress in stiffener direction
k_{sp}	=	factor for additional stresses in asymmetric profile sections, see Sec.3 Table 5
Y_m	=	1.1.

Table 4 Factor k_{sp} for various sections

Type of section	k_{sp}
Flat bars and symmetric T-sections	1.00
Bulb sections	1.03
Asymmetric T-sections $b_2/b_1 \approx 0.5$	1.05
Rolled angles [L-sections]	1.15

The actual sectional properties shall be related to an axis parallel to the attached plating.

Note:

For asymmetric sections the additional stress σ_h occurring in asymmetric sections according to Figure 1 may be determined by the following equation:

$$\sigma_h = Q \cdot \ell_f \cdot t_f \cdot \frac{b_1^2 - b_2^2}{c \cdot W_y \cdot W_z} \text{ [MPa]}$$

where:

Q	=	load on section parallel to its web within the unsupported span ℓ_f [kN], defined as
	=	$p \cdot a \cdot \ell_f$ in case of uniformly distributed load p [kPa]
ℓ_f	=	unsupported span of flange [m]
t_f, b_1	=	flange dimensions [mm]
b_2	=	as shown in Figure 7

$b_1 \geq b_2$

W_Y	=	elastic section modulus of section related to the y-y axis including the effective width of plating [cm^3]
W_z	=	elastic section modulus of the partial section consisting of flange and half of web area related to the z-z axis [cm^3], (bulb sections may be converted into a similar L-section).
c	=	factor depending on kind of load, stiffness of the section's web and length and kind of support of profile For profiles clamped at both ends c = 80 can be taken for approximation. A precise calculation may be required, e.g. for longitudinal frames.

This additional stress σ_h shall be added directly to other stresses such as those resulting from local and hull girder bending.

The total stress [MPa] according to local bending thus results in:

$$\sigma = \frac{Q \cdot \ell_f \cdot 1000}{12 \cdot W_y} \cdot \left[1 + \frac{12 \cdot t_f \cdot \left(b_1^2 - b_2^2\right)}{1000 \cdot c \cdot W_z}\right]$$

The required elastic section modulus W_y shall be multiplied with the factor k_{sp} depending on the type of profile and the boundary conditions expressed by the factor c.

$$k_{sp} = \frac{12 \cdot W_y}{Q \cdot \ell_f \cdot 1000} \cdot \sigma$$

---e-n-d---o-f---n-o-t-e---

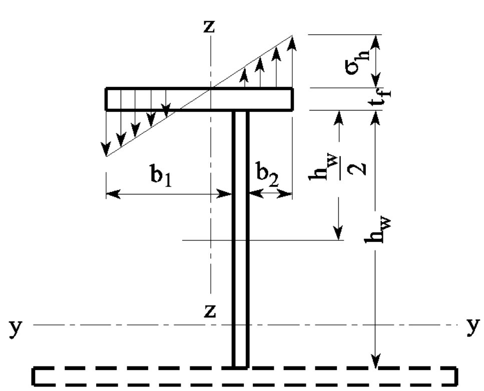


Figure 1 Additional stresses at asymmetric sections

6.2.3 Beam analysis

The maximum normal bending stress, σ and shear stress, τ in a stiffener using properties with reduced end fixity, variable load or being part of grillage shall be determined by direct calculations taking into account:

- the distribution of static and dynamic pressures and forces, if any
- the number and position of intermediate supports (e.g. decks, girders, etc.)
- the condition of fixity at the ends of the stiffener and at intermediate supports
- the geometrical characteristics of the stiffener on the intermediate spans.

The equivalent stresses σ_v resulting from σ_L and τ_L shall not exceed the following value:

$$\sigma_v = \sqrt{\sigma^2 + 3\tau^2} \leq \frac{f_m R_y}{\gamma_m f_L}$$

$\gamma_m = 1.1$

6.2.4 Standard dimensioning method

The scantlings of lateral loaded secondary and primary members can be determined on the basis of the following criteria:

- If the cross section is only loaded by shear, the following simplified criterion can be applied (buckling due to shear has to be checked additionally):

$$\frac{F_s \cdot \gamma \cdot y_m}{Q_u} \leq 1.0$$

- If combined loads are applied, the following combined criterion for simultaneous bending, shear and/or axial force has to be applied:

$$\frac{N}{N_{ur}} + \frac{\gamma \cdot M_b + N \cdot e_N}{M_{ur}} \leq 1.0$$

where:

N = sum of local and global forces [kN], in longitudinal direction of structural element
N_x = local axial force [kN]
= $\gamma \cdot N_x + \frac{\sigma_L}{10} \cdot A_{eff}$
N_{ur} = normal force capacity reduced due to shear [kN]
= $N_u \cdot \kappa_s$
Qu = shear capacity [kN]
= $\frac{1}{\sqrt{300}} \cdot A_{shr} \cdot R_y \cdot f_m$
Mu = bending capacity [kNm]
= $\frac{W_{el}}{1000} \cdot \frac{R_y \cdot f_m}{\gamma_m \cdot k_{sp}}$
M_{ur} = bending capacity reduced due to shear [kNm]
= $M_u \cdot \kappa_s$
e_N = vertical shift of the main axis of the effective area Aeff related to the cross-section A
k_s = shear reduce factor
= $1 - \left(\frac{5 \cdot \gamma}{4 \cdot Q_u} - \frac{1}{4}\right)^2$

$= \quad 1.0 \quad if \frac{\gamma}{Q_u} \leq 0.2$

6.3 Alternative scantling verification

6.3.1 Application

The strength assessment in this paragraph is based on the application of ultimate strength formulations is using the theory of plasticity and the yield line theory. The material behaviour for these calculations was assumed to be elastic perfectly plastic (without any hardening effects). Paragraph is based on the application of ultimate strength formulations using the theory of plasticity and the yield line theory. The material behaviour for these calculations was assumed to be elastic perfectly plastic (without any hardening effects).

The ultimate strength defined as the maximum normal force, shear and/or bending capacity beyond which the girder will collapse and shall calculated with the following limitations:

— yielding of material up to a maximum strain of 10%
— buckling of compressed structural elements including nominal pre-deflection as described in App.B.
— non-linear deformation effects
— sufficient rotation capacity in way of determined plastic hinges according to [6.3.2].

The use of ultimate strength criteria shall be reduced to those structural members where a permanent deformation (plasticized sections) is acceptable for a (temporarily, at least) safe operation of the vessel (e.g. watertight bulkheads).

6.3.2 Rotation capacity

Care shall be taken at the plastic hinges location. In general, following design principles shall be fulfilled at plastic hinges within the longitudinal extents of about twice of the height of the structural member: twice of the height of the structural member:

— constant scantlings of the profile
— no cut-outs, holes, discontinuities or notches
— no welds at flanges in local transverse direction on the tension side of the member.

For aluminium profiles the rotation capacity is generally limited by the ultimate strain at the outer edge(s) (in opposite to steel structures). The elastic beam loads according to Table 3 should be used instead of the ultimate loads.

(Mono-) symmetric profiles are preferably provided. For asymmetrical flanges additional tripping brackets shall be provided at plastic hinge within web depth (for aluminium structure with a minimum distance of 30 mm to the hinge).

A limitation of bending and rotation capacity due to buckling according to App.B is not expected for cross-sections with following guidance values for the ratio web depth to web thickness and/or flange breadth to flange thickness. Subject to hole and cut-outs they can form a plastic hinge with the rotation capacity required for plastic analysis without reduction of the resistance:

— flat bars: $\frac{h_w}{t_w} \leq 9 \cdot \epsilon$
— angle, tee and bulb sections:

web: $\frac{h_w}{t_w} \leq 33 \cdot \epsilon$

flange: $\frac{b_i}{t_f} \leq 9 \cdot \epsilon$

ε = $\epsilon_{St} = \sqrt{\frac{235}{R_y}}$ for steel

ε = $\epsilon_{Al} = \sqrt{\frac{40}{R_y}}$ for aluminium

Used scantlings are defined according to App.B Figure 2.

6.3.3 Ultimate nominal loads

If all requirements of [6.3.1] and [6.3.2] are fulfilled, the nominal ultimate shear forces and bending moments according to Table 3 can be taken into account for the general case of trapezoidal loaded beams with constant cross sections and typical end connection.

For the concentrated loads and typical end connection, the following nominal ultimate values for F_s and M_b can be used for primary supporting members and secondary stiffening members:

$$F_s = F_E \text{ [kN]}$$

$$M_b = \frac{F_E \cdot \ell}{4 + 2 \cdot j} \cdot \left(1 - \frac{\ell_E}{2 \cdot \ell}\right) \text{ [kNm]}$$

where:

ℓ = unsupported length [m]

ℓ_E = length of application of the force F_E;

= 0.0 if ℓ_E is not known

$\ell_E < \ell$

j = see Sec.4 [4].

This equation is based on the assumption that the ultimate bending capacity M_p has been calculated considering the maximum shear force F_E.

For other loads and end connections F_s and M_b shall be calculated separately.

6.3.4 Ultimate strength

The scantlings of lateral loaded stiffeners and primary supporting members can be determined on the basis of the standard dimensioning method according to [4.2.2].

If the cross section fulfils the requirements given in [6.3.1] and [6.3.2], the bending capacity M_u in kNm, can be determined as follows:

$$M_u = \frac{W_p}{1000} \cdot \frac{R_y \cdot f_m}{\gamma_m}$$

where:

γ_m = 1.1

For the determination of W_{pl} in the equations above, the effective width of the plating according to App.A [7.2] has to be considered.

7 Pillars

F = pillar load in kN

= $p_D \cdot A + F_i$

p_D = load on decks [kPa] according to Ch.3 Sec.3 [3.4]
A = load area supported by the pillar [m^2]
F_i = load from pillars located in line, above the pillar considered [kN]
ℓ_s = length of the pillar [cm]
I_S = moment of inertia of the pillar [cm^4] under consideration of effective width of each plate field according to App.B [2.2]
A_{eff} = effective sectional area of the pillar [cm^2]
A_H = effective sectional area of heads and heels considering hot spots [cm^2].

7.1 General

7.1.1 Pillars shall be fitted in the same vertical line wherever possible. If not possible, effective means shall be provided for transmitting their loads to the supports below. Effective arrangements shall be made to distribute the load at the heads and heels of all pillars. Where pillars support eccentric loads, they shall be strengthened for the additional bending moment imposed upon them.

7.1.2 Pillars shall be provided in line with double bottom girders and/or floors or as close thereto as practicable, and the structure above and below the pillars shall be of sufficient strength to provide effective distribution of the load. Where pillars connected to the inner bottom are not located in way of the intersection of floors and girders, partial floors or girders or equivalent structures shall be fitted as necessary to support the pillars.

7.1.3 Tubular pillars are not permitted in tanks for flammable liquids. Steel pillars provided in tanks shall be of solid or open section type.

7.1.4 Connections
Heads and heels of pillars shall be secured by thick doubling plates and brackets as necessary. Where the pillars are likely to be subjected to tensile loads, the head and heel of pillars shall be efficiently secured to withstand the tensile loads and the doubling plates replaced by insert plate.

In general, the thickness of doubling plates shall be not less than 1.5 times the thickness of the pillar. Pillars shall be attached at their heads and heels by continuous welding.

Doubling plates are neither permitted in tanks for steel plates nor in tanks for flammable liquids, in general.

7.2 Buckling criterion

7.2.1
The chosen scantlings of a pillar have to meet the following buckling criterion:

$$\frac{\sigma_x}{\kappa} \leq \frac{R_y}{\gamma_m}$$

where:

σ_x = stress in longitudinal direction of the pillar, in MPa

= $F \cdot \frac{10}{A_{eff}}$

κ = reduction factor

= $\frac{1}{\Phi + \sqrt{\Phi^2 - \lambda^2}}$

Φ = $0.5 \cdot \left(1 + n_p \cdot \left(\lambda - \lambda_0\right) + \lambda^2\right)$

ℓ_s = $\sqrt{\frac{R_y}{\sigma_{ki}}} \geq \lambda_0$

σ_{ki} = critical stress in MPa

= $\frac{\pi^2 \cdot E \cdot l_s}{l_s^2 \cdot k_s^2 \cdot A_{eff}}$

k_S = 1.0 in general

= 0.7 for pillars with constrained ends which are supported in direction perpendicular to the axis, if agreed by the Society in each single case.

γ_m = 1.66

for steel:

λ_0 = 0.2

n_P = 0.34 for pipes and box sections

= 0.49 for open sections

for aluminium without heat treatment (i.e. 5000 series):

λ_0 = 0.0

n_p = 0.32

for aluminium with heat treatment (i.e. 6000 series):

λ_0 = 0.1

n_p = 0.20.

7.2.2

For pillars which are additionally loaded by bending moments the nominal bending stress shall be determined by multiplying the bending stress σ_b with the following magnification factor c_{ki}:

$$c_{ki} = \frac{1}{1 - \frac{F}{F_{ki}}}$$

where:

F_{ki} = $\frac{\pi^2 \cdot l_s \cdot E}{l^2}$

The bending stress increased by the magnification factor c_{ki} shall not exceed the following condition:

$$\sigma_b \cdot c_{ki} \leq \frac{f_{m \cdot} R_y}{\gamma_m}$$

γ_m = 1.66

7.3 Heads and heels

Structural members at heads and heels of pillars as well as substructures shall be constructed according to the forces they are subjected to. The connection shall be so designed that the following condition is met:

$$\frac{10 \cdot F}{A_H} \leq \frac{f_m \cdot R_y}{\gamma_m}$$

γ_m = 1.66

Where pillars are affected by tension loads and/or bending moments doublings are not permitted.

SECTION 4 STRUCTURAL ARRANGEMENTS

1 Bottom and shell

1.1 General

1.1.1 The scantlings of plating, secondary and primary members of the bottom, double bottom and shell structures shall be determined according to Sec.3 [5] and [4].

1.1.2 The plate thicknesses shall be tapered gradually, if different. Gradual taper is also to be effected between the thicknesses required for local strengthening.

1.1.3 When the bottom or inner bottom is longitudinally stiffened, the longitudinals shall in general be continuous through transverse members within 0.8*L* amidship.

1.1.4 Any variation in the height of the double bottom is generally to be made gradually and over an adequate length. The knuckles of inner bottom plating shall be located in way of plate floors. Where such arrangement is not possible, suitable longitudinal structures such as partial girders, longitudinal brackets, fitted across the knuckle shall be arranged.

1.1.5 Striking plates of adequate thickness or other equivalent arrangements shall be provided under sounding pipes to prevent the sounding rod from damaging the plating.

1.1.6 The number and size of manholes in floors located in way of seatings and adjacent areas shall be kept to the minimum necessary for double bottom access and maintenance.

In general, manhole edges shall be stiffened with flanges. Failing this, the floor plate shall be adequately stiffened with flat bars at manhole sides.

1.1.7 If applicable (e.g. for double bottom), the requirements of [2] (tanks), [3] (decks) and [4] (bulkheads) shall be fulfilled.

1.2 Arrangement of double bottom

1.2.1 It is recommended that a double bottom shall be fitted extending from the collision bulkhead to the afterpeak bulkhead, as far as this is practicable and compatible with the design and service of the ship.

1.2.2 The double bottom height at the centreline, irrespective of the location of the machinery space, shall be not less than the value defined in Ch.2 Sec.2 [2.2].

1.2.3 If a double bottom is provided the inner bottom shall be extended to the ship's sides as to protect the bottom up to the turn of the bilge.

1.2.4 In fore- and afterpeak a double bottom need not be arranged.

1.3 Plating

1.3.1 Keel

Keel plating shall extend over the flat of bottom for the full length of the ship.

For the design of a keel the docking operating conditions according to [1.8] shall be observed.

Flat keel

— the width of the flat keel, in m, shall not be less than 0.8 + L/200, without being taken greater than 2.3 m
— the thickness of the flat plate keel shall not be less than:

$$t_{FK} = t_B + 1.5 \text{ [mm]}$$

t_B = required thickness of bottom plating [mm] according to Sec.3 [5].

Box keel

— the bottom plate of the box keel shall have a plate thickness not less than the flat keel thickness t_B + 1.5 mm
— the side (skeg) plates shall have a thickness not less than t_B + 1.0 mm
— sufficient brackets in line (horizontal and/or vertical) shall be provided at the cross point of the bottom plating and side plating of the box keel in way of floors.

Bar keel

— the sectional area A of the bar keel shall not be less than:

$$A = 0.5 \cdot L \text{ [cm}^2\text{]}$$

— where a bar keel is arranged, the adjacent garboard strake shall have the scantlings of a flat plate keel.

1.3.2 Sheerstrake

The width of the sheerstrake, in m, shall not be less than 0.8 + L/200, without being taken greater than 1.8 m.

The thickness of the sheerstrake shall, in general, not be less than the greater of the following two values:

$$t = 0.5 \cdot \left(t_D + t_s\right) \text{ [mm]}$$

where:

t = t_s
t_D = required thickness of strength deck according to Sec.3 [5]
t_S = required thickness of side shell according to Sec.3 [5].

Where the connection of the deck stringer with the sheerstrake is rounded, the radius shall be at least 15 times the plate thickness or 150 mm whichever is greater.

The transition from a rounded sheer strake to an angled sheer strake associated with the arrangement of superstructures shall be designed to avoid any discontinuities.

Welds on upper edge of sheerstrake are subject to special approval.

Regarding welding between sheerstrake and deck stringer, see [3.3].

Holes for scuppers and other openings shall be smoothly rounded. Notch factors, see App.A [8.5].

1.4 Stiffeners

1.4.1 Transverse framing

The lower bracket attachment to the bottom structure shall be determined on the basis of the frame section modulus.

The upper bracket attachment to the deck structure and/or to the tween deck frames shall be determined on the basis of the section modulus of the deck beams or tween deck frames whichever is the greater.

Where the bottom is framed longitudinally but the sides are framed transversely, flanged brackets having a thickness of the floors shall be fitted between the plate floors at every transverse frame, extending to the outer longitudinals at the bottom and inner bottom.

Where frames are supported by a longitudinally framed deck, the frames fitted between web frames shall be connected to the adjacent longitudinals by brackets. The scantlings of the brackets shall be determined on the basis of the section modulus of the frames.

1.4.2 Longitudinal framing

Longitudinals shall preferably be continuous through floor plates and/or transverses. Attachments of their webs to the floor plates and transverses shall be sufficient to transfer the support forces.

Forward of 0.1 L from FP webs of longitudinals shall be connected effectively at both sides to transverse members. If the flare angle α exceeds 40° additional heel stiffeners or brackets shall be arranged.

Where longitudinals are not continuous at watertight floors and bulkheads, they shall be attached to the floors by brackets of the thickness of plate floors, and with a length of weld at the longitudinals equal to 2 × depth of the bottom longitudinals.

Where necessary, for longitudinals between transverse bulkheads and side transverses additional stresses resulting from the deformation of the side transverses shall be taken into account.

In the fore body, where the flare angle α exceeds 40° and in the aft body where the flare angle exceeds 75° the unsupported span of the longitudinals located between $T_{min} - c_0$ and $T + c_0$ shall not be larger than 2.6 m. Otherwise tripping brackets shall be arranged.

1.4.3 Strengthening in fore and aft body

Fore and aft body shall be properly designed considering the hydrodynamic pressure defined in Ch.3 Sec.3 [3.2].

Between the point of the greatest breadth of the ship at maximum draft and the collision bulkhead tripping brackets shall be fitted if frames are not supported over a distance exceeding 2.6 m.

1.4.4 Struts

The cross sectional area of the struts shall be determined according to Sec.3 [7] analogously. The strut load shall be taken from the direct stress analysis.

1.5 Primary members

1.5.1 Bottom centre girder

All ships shall have a centre girder or two longitudinal girders near to each other for docking.

The centre girder has to be extended as far forward and aft as practicable. It shall be connected to the girders of a non-continuous double bottom or shall be scarphed into the double bottom by two frame spacings.

Towards the ends the thickness of the web plate as well as the sectional area of the top plate may be reduced by 10 per cent. Lightening holes shall be avoided.

Lightening holes in the centre girder are generally permitted only outside 0.75 L amidships. Their depth shall not exceed half the depth of the centre girder and their lengths shall not exceed the half frame spacing.

No centre girder is required in way of engine seating in case of centre engine.

1.5.2 Bottom side girders

The side girders shall extend as far forward and aft as practicable. They shall be connected to the girders of a non-continuous double bottom or shall be scarphed into the double bottom by two frame spacing.

Towards the ends, the web thickness and the sectional area of the face plate may be reduced by 10 per cent.

At least one side girder shall be fitted in the engine room and in way of 0.25 *L* aft of *FP*. The actual number of bottom side girders in all parts of double bottom has to be arranged at distances following from the overall bottom analyses.

1.5.3 Docking brackets

Docking brackets connecting the centreline girder to the bottom plating, shall be connected to the adjacent bottom longitudinals and supporting plates on the centre girder.

1.5.4 Margin plates

The margin plate has to be watertight. Brackets in line with floor plates and frames shall be provided to connect the margin plate to the side framing.

In case of longitudinal framing system stiffening plates shall be provided at the margin plate to connect the margin plate to the longitudinals in the double bottom.

1.5.5 Floor plates

For the connection of floor plates with the frames, see App.C.

Deep floors, particularly in the after peak, shall be provided with buckling stiffeners.

The floor plates shall be provided with limbers to permit the water to reach the pump suctions.

In ships having a considerable rise of floor, the depth of the floor plate webs at the beginning of the turn of bilge shall not be less than the depth of the frame.

The face plates of the floor plates shall be continuous over their span. If they are interrupted at the centre keelson, they shall be connected to the centre keelson by means of full penetration welding.

Where longitudinal framing system changes to transverse framing system, structural continuity or sufficient scarphing shall be provided for.

Plate floors:

— the spacing of plate floors will result from the overall analysis according to Sec.3 [6]
— plate floors shall be fitted:
  — in the engine room, as far as necessary
  — under bulkheads
  — under corrugated bulkheads, see also [4.3]
— where the longitudinal framing system is adopted, the floor spacing should, in general, not exceed 5 times the longitudinal frame spacing
— in way of strengthening of bottom forward, the plate floors shall be connected to the shell plating and inner bottom by continuous fillet welding.

Bracket floors:

— where plate floors are not required according to [1.5.5] bracket floors may be fitted
— bracket floors consist of bottom frames at the shell plating and reversed frames at the inner bottom, attached to centre girder, side girders and ship's side by means of brackets.

Floor plates in the peaks:

— the thickness of the floor plates in the peaks shall be determined according to the direct analysis
— the floor plates in the afterpeak shall extend over the stern tube, see also [1.6.2]
— where propeller revolutions are exceeding 300 rpm (approx.) the peak floors above the propeller shall be strengthened. Particularly in case of flat bottoms additional longitudinal stiffeners shall be fitted above or forward of the propeller.

1.5.6 Web frames and stringers

Side transverses:

— in the fore body where flare angles α are larger than 40° the web shall be stiffened in the transition zone to the deck transverse.

Web frames in machinery spaces:

— in the engine rooms, web frames suitably spaced shall be fitted. Generally, they should extend up to the uppermost continuous deck
— for combustion engines web frames shall generally be fitted at the forward and aft ends of the engine. The web frames shall be evenly distributed along the length of the engine
— where combustion engines are fitted aft, stringers spaced 2.6 m apart shall be fitted in the engine room, in alignment with the stringers in the after peak, if any.

1.6 Stem and sternframe structures

1.6.1 Plate stem and bulbous bow

The thickness of welded plate stems shall be determined according to Sec.3 [5]. The design pressure, in kPa, acting on the stem can be calculated as follows:

$$p_{BB} = 1.85 \cdot \left(0.28 \cdot v_0 + 0.82 \cdot \sqrt{L}\right)^2$$

The plate thickness shall not be less than the required thickness according to [1.3].

The extension ℓ of the stem plate from its trailing edge aftwards shall not be smaller than:

$$\ell \; = 70 \cdot \sqrt{L} \text{ [mm]}$$

Starting from 600 mm above the load waterline up to $T + c_0$, the thickness may gradually be reduced to 0.8 t or the thickness according to Sec.3, whichever is greater.

Plate stems and bulbous bows shall be stiffened as shown in Sec.3 Figure 6.

Note:

Large bulbous bows may be subjected to horizontal dynamic pressure p_{Sdyn}, acting from one side only, see Ch.3 Sec.3 [3.2].

For the effective area of p_{Sdyn}, the projected area of the z-x-plane from forward to the collision bulkhead may be assumed.

---e-n-d---o-f---n-o-t-e---

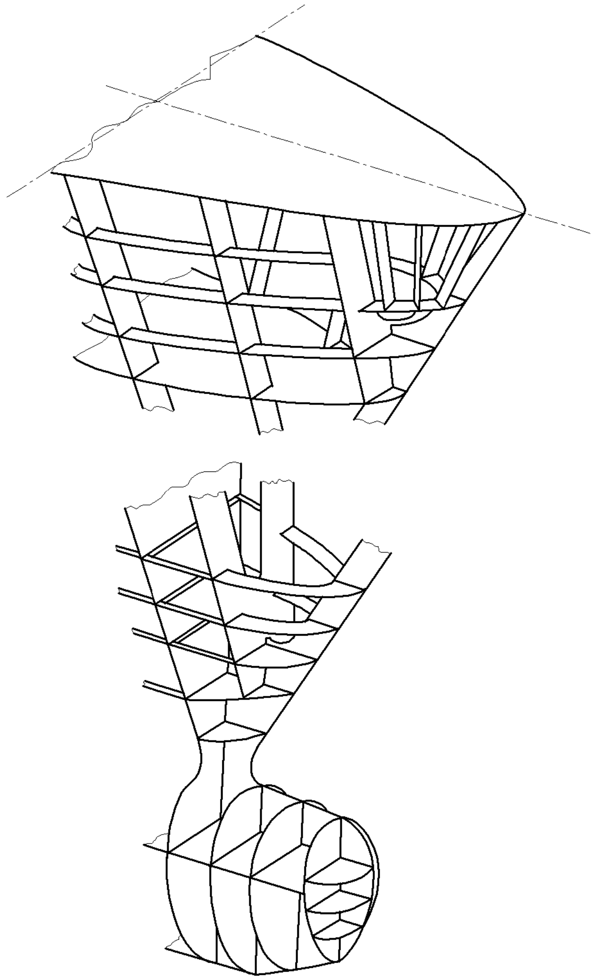


Figure 1 Arrangement of fore hooks and cant frames at the bow

1.6.2 Sternframe

The plate thickness shall not be less than the required thickness according to [1.3].

The aft structure in way of the propeller has to be investigated for the enforced vibrations by the propeller.

The stern tube shall be sufficiently supported by the ship's stern structure.

Where either the stern overhang is very large or the maximum breadth of the space divided by watertight and wash bulkheads is greater than 20 m, additional longitudinal wash bulkheads may be required.

Floors shall be fitted at each frame space in the aft peak and carried to a height at least above the stern tube. Where floors do not extend to flats or decks, they shall be stiffened by flanges at their upper end.

1.7 Sea chests

The scantlings of sea chests shall be determined according to Sec.3 [5] using the pressure, in kPa:

$$p_{sc} = 125 \cdot p_v$$

p_v = blow out pressure in bar, at the safety valve.

p_v shall not be less than 2 bar, see also Pt.4 Ch.6.

The sea-water inlet opening edges in the shell shall be stiffened related to the stress level. The openings shall be protected by gratings.

A cathodic corrosion protection with galvanic anodes made of zinc or aluminium shall be provided in sea chests with chest coolers. For the suitably coated plates a current density of 30 μA/m^2 shall be provided and for the cooling area a current density of 180 μA/m^2. For details see the Society's guidelines for corrosion protection and coating systems.

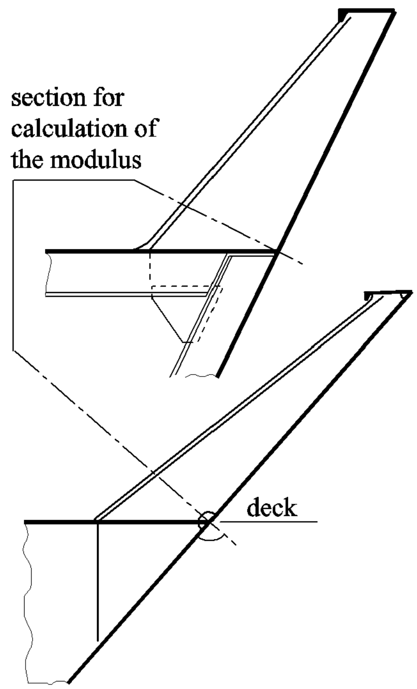


Figure 2 Inclined bulwark stays

1.8 Docking

1.8.1 General

For ships exceeding 120 m in length, for ships of special design, particularly in the aft body and for ships with a docking load of more than 700 kN/m a special calculation of the docking forces is required.

The proof of sufficient strength can be performed either by a simplified docking calculation or by a direct docking calculation. The number and arrangement of the keel blocks shall comply with the submitted docking plan.

The partial safety factor for material resistance γ_M shall be used with 1.05.

Direct calculations are required for ships with unusual overhangs at the ends or with inhomogeneous keel block distribution.

1.8.2 Simplified docking calculation

The local forces of the keel blocks acting on the bottom structures can be calculated in a simplified manner using the nominal keel block load q_0. Based on these forces sufficient strength shall be shown for all structural bottom elements which may be influenced by the keel block forces. The nominal keel block load q_0 is calculated as follows, see also Sec.3 Figure 2:

$$q_0 = \frac{g \cdot \Delta_{dock} \cdot C}{L_{KB}} \text{ [kN/m]}$$

where:

Δ_{dock} = ship weight during docking [t]
L_{KB} = length of the keel block range [m]; i.e. in general the length of the horizontal flat keel
C = weighting factor
= 1.25 in general
= 2.0 in the following areas:

- within 0.075 L_{KB} from both ends of the length L_{KB}
- below the main engine
- in way of the transverse bulkheads along a distance of 2 – e

e = distance of plate floors adjacent to the transverse bulkheads [m]; for e no value larger than 1 m needs to be taken.

If a longitudinal framing system is used in the double bottom in combination with a centre line girder it may be assumed that the centre line girder carries 50% of the force and the two adjacent keel block longitudinals 25% each.

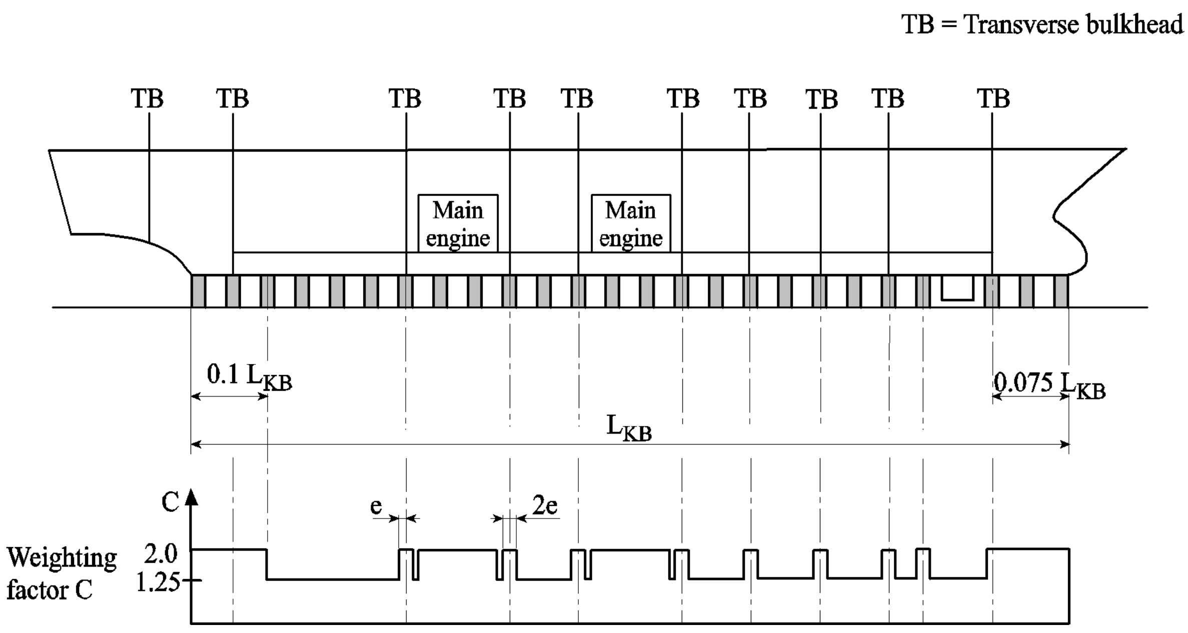


Figure 3 Definition of weighting factor *C*

1.8.3 Direct docking calculation

If the docking block forces are determined by direct calculation, e.g. by a finite element calculation, considering the stiffness of the ship's body and the weight distribution, the ship has to be assumed as elastically bedded at the keel blocks. The stiffness of the keel blocks has to be determined including the wood layers. If a floating dock is used, the stiffness of the floating dock shall be taken into consideration.

1.8.4 Permissible stresses

The equivalent stresses σ_v shall not exceed following value:

$$\sigma_V \leq \frac{f_m \cdot R_y}{\gamma_m}$$

$\gamma_m = 1.05$

1.8.5 Buckling strength

The bottom structures shall be examined according to App.B. For this purpose a safety factor γ_m = 1.05 shall be taken into account.

2 Tank

2.1 General

2.1.1 Subdivision of tanks

In tanks extending over the full breadth of the ship intended to be used for partial filling, (e.g. oil fuel and fresh water tanks), at least one longitudinal bulkhead shall be fitted, which may be a swash bulkhead.

Where the forepeak is intended to be used as tank, at least one complete or partial longitudinal swash bulkhead shall be fitted, if the tank breadth exceeds 0.5 *B* or 6 m, whichever is the greater.

When the afterpeak is intended to be used as tank, at least one complete or partial longitudinal swash bulkhead shall be fitted. The largest breadth of the liquid surface should not exceed 0.3 *B* in the aft peak.

Peak tanks exceeding 0.06 *L* or 6 m in length, whichever is greater, shall be provided with a transverse swash bulkhead.

2.1.2 Air, overflow and sounding pipes

Each tank shall be fitted with air pipes, overflow pipes and sounding pipes. The air pipes shall be led to above the exposed deck. The arrangement shall be such as to allow complete filling of the tanks. See also Pt.4 Ch.6.

The sounding pipes shall be led to the bottom of the tanks, see also Pt.4 Ch.6.

2.1.3 Forepeak tank

Oil shall not be carried in a forepeak tank.

2.1.4 Separation of oil fuel tanks from tanks of other liquids

Oil fuel tanks shall be separated from tanks for lubricating oil, hydraulic oil and potable water by cofferdams.

Upon special approval on small ships the arrangement of cofferdams between oil fuel and lubricating oil tanks may be dispensed with provided that:

— the common boundary is continuous, i.e. it does not abut at the adjacent tank boundaries, see Figure 5.

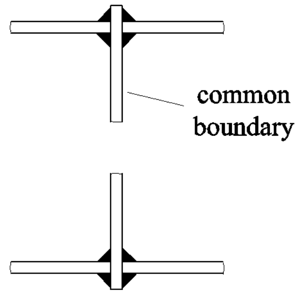


Figure 4 Welding at not continuous tank boundaries

— where the common boundary cannot be constructed continuously according to Figure 5, the fillet welds on both sides of the common boundary shall be welded in two layers and the throat thickness shall not be less than 0.5 × t (t = plate thickness)
— stiffeners or pipes do not penetrate the common boundary
— the corrosion addition t_c for the common boundary is not less than 1.0 mm
— common boundaries and the above mentioned details are clearly indicated in the drawings submitted for approval.

Fuel oil tanks adjacent to lubricating oil circulation tanks are subject to the provisions of Pt.4 Ch.6 in addition to the requirements stipulated in [2.1.4] above.

2.1.5 Double bottom tanks

Where practicable, lubricating oil discharge tanks or circulating tanks shall be separated from the shell.

Manholes for access to fuel oil double bottom tanks situated under oil tanks for the supply of other ships are neither permitted in these supply tanks nor in the engine rooms.

2.1.6 Potable water tanks

Potable water tanks shall be separated from tanks containing liquids other than potable water, ballast water, distillate or feed water.

In no case sanitary arrangement or corresponding piping shall be fitted directly above the potable water tanks.

Manholes arranged in the tank top shall have sills.

If pipes carrying liquids other than potable water shall be led through potable water tanks, they shall be fitted in a pipe tunnel.

Air and overflow pipes of potable water tanks shall be separated from pipes of other tanks.

2.1.7 Cross references

For pumping and piping, see also Pt.4 Ch.6.

Where tanks are provided with cross flooding arrangements the increase of the pressure head shall be taken into consideration.

2.2 Scantlings

2.2.1 General

The scantlings of plating, secondary and primary members of the boundaries of tanks shall be determined according to Sec.3 [5] and [4].

For longitudinal bulkheads the design stresses according to and the stresses due to local loads shall be considered.

For inclined bulkheads with an inclination to the horizontal of less than or equal 45° see [3].

If the boundaries of tanks are formed by integrated structural elements of the ship, their dimensioning has to follow the relevant requirements, in addition.

Detached tanks, which are independent from the ship's structure, shall be designed according to [2.3].

2.2.2 Plating

The minimum thickness is 3.0 mm, for the corrosion addition see Sec.3 [5.1].

2.2.3 Stiffeners and girders

The buckling strength of the webs shall be checked according to App.B.

The section moduli and shear areas of horizontal stiffeners and girders shall be determined according to Sec.3 Table 3.

2.3 Detached tanks

2.3.1 Plating

The minimum thickness is 3.0 mm, for the corrosion addition see Sec.3 [5.1]. For stainless steel the minimum thickness can be reduced to 2.5 mm.

For corrugated tank walls see [4.3].

Special consideration has to be given to the avoidance of vibrations.

2.3.2 Arrangement

Detached tanks shall be adequately secured against forces due to the ship's motions. The forces can be calculated considering the acceleration components given Ch.3 Sec.3 [1.2].

Detached tanks shall be provided with anti-floatation devices. It shall be assumed that the flooding reaches the design water line, for ships with proven damage stability the extreme damage waterline. The stresses in the anti-floatation devices caused by the floatation forces shall not exceed the material's yield stress.

Fittings and piping on detached tanks shall be protected by battens. Gutterways shall be fitted on the outside of tanks for draining any leakage oil.

3 Decks and walls

3.1 General

The scantlings of plating, secondary and primary members of the decks and walls shall be determined according to Sec.3 [5] and [4].

The loads as defined in Ch.3 Sec.3 [3.2] shall be considered, if applicable.

Inclined walls with an inclination to the horizontal of less than or equal 45° will be treated as deck, see Sec.3 Figure 6.

Vertical and horizontal watertight bulkheads and non-watertight partitions shall be designed in accordance with [4].

If applicable, the requirements of [2] (tanks) shall be fulfilled.

3.2 Superstructures and deckhouses

Superstructures with longitudinal walls immediately besides the shell are subjected – even for a very restricted length – to the same elongation as the hull. Therefore at the ends of these structures high longitudinal stresses as well as shear stresses are transferred to the longitudinal walls of superstructures and deckhouses. Fatigue strength investigations have to be made and submitted for approval.

3.3 Plating

Where sheathing is used, attention shall be paid that the sheathing does not affect the steel. The sheathing shall be effectively fitted to the deck. Deformations of the deck plating have to be considered.

If the thickness of the strength deck plating is less than that of the side shell plating, a stringer plate shall be fitted having the width of the sheerstrake and the thickness of the side shell plating, see [1.3.2].

The welded connection between strength deck and sheerstrake may be effected by fillet welds according to App.C Table 3. Where the plate thickness exceeds approximately 25 mm, a double bevel weld connection according to App.C [2.3.2] shall be provided for instead of fillet welds. Bevelling of the deck stringer to 0.65 times of its thickness in way of the welded connection is admissible. In special cases a double bevel weld connection may also be required, where the plate thickness is less than 25 mm.

The deck structure inside line of openings shall be so designed that the compressive stresses acting in the ship's transverse direction can be safely transmitted. Proof of buckling strength shall be provided according to App.B.

Areas of structural discontinuities, e.g. at end of superstructures, have to be carefully designed and analysed. The strength deck plating shall be sufficiently extended into a superstructure. As a guidance the thickness of the superstructure side plating as well as the strength deck in a breadth of 0.1 B from the shell shall be strengthened by about 20 per cent. The strengthening shall extend over a region of about twice of the height of the end wall abaft to forward.

3.4 Secondary stiffeners

When the strength deck is longitudinally stiffened, the longitudinals shall in general be continuous at transverse members within 0.8L.

Transverse deck beams shall be connected to the frames by brackets according to App.A [8.4.3].

Deck beams may be attached to girders beside hatch openings by double fillet welds where there is no constraint. The length of weld shall not be less than 0.6 times the depth of the section.

Where deck beams shall be attached to girders of considerable rigidity (e.g. box girders), brackets shall be provided.

Regarding the connection of deck longitudinals to transverses and bulkheads, [1.4.2] shall be observed.

3.5 Primary members

Face plates shall be stiffened by tripping brackets. At girders having symmetrical face plates, the tripping brackets shall be arranged alternately on both sides of the web.

End attachments of primary members in way of supports shall be so designed that the bending moments and shear forces can be transferred. Bulkhead stiffeners under girders shall be sufficiently dimensioned to support the girders.

Below strength decks girders shall be fitted in alignment with longitudinal walls of superstructures and deckhouses above, which shall extend at least over three frame spacings beyond the end points of the longitudinal walls. The girders shall overlap with the longitudinal walls by at least two frame spacings.

3.6 Elastic mounting of deckhouses

3.6.1 General

The elastic mountings shall be type approved by the Society. The stresses acting in the mountings which have been determined by calculation shall be proved by means of prototype testing on testing machines. Determination of the grade of insulation for transmission of vibrations between hull and deckhouses is not part of this type approval.

The height of the mounting system shall be such that the space between deck and deckhouse bottom remains accessible for repair, maintenance and inspection purposes. The height of this space shall normally not be less than 600 mm.

For the fixed part of the deckhouse on the weather deck, a coaming height of 380 mm shall be observed, as required for coamings of doors in superstructures which do not have access openings to under deck spaces.

For pipelines, see Pt.4 Ch.6.

Electric cables shall be fitted in bends in order to facilitate the movement. The minimum bending radius prescribed for the respective cable shall be observed. Cable glands shall be watertight. For further details, see Pt.4 Ch.8.

The following scantling requirements for rails, mountings, securing devices, stoppers and substructures in the hull and the deckhouse bottom apply to yachts in unrestricted service. For special yachtss and for yachts intended to operate in restricted service ranges requirements differing from those given below may be applied.

3.6.2 Design loads

For scantling purposes the following design loads apply:

Weight:

- the weight *G* induced loads result from the weight of the fully equipped deckhouse, considering also the acceleration due to gravity and the acceleration due to the ship's movement in the seaway. The weight induced loads shall be assumed to act in the centre of gravity of the deckhouse
- the individual dimensionless accelerations a_z (vertically), a_y (transversely) and a_x (longitudinally) and the dimensionless resultant acceleration shall be determined according to Ch.3 Sec.3 for ψ = 1.0 and f_Q = 1.0
- the support forces in the vertical and horizontal directions shall be determined for the various angles. The scantlings shall be determined for the respective maximum values.

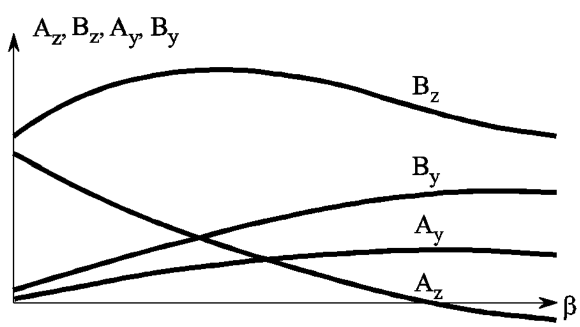


Figure 5 Support forces

Water pressure and wind pressure:

- the water load due to the wash of the sea is assumed to be acting on the front wall in the longitudinal direction only. The design load is:
- the water pressure shall not be less than:
  - p_S at the lower edge of the front wall
  - 0 at the level of the first tier above the deckhouse bottom
- the design wind load acting on the front wall and on the side walls shall not be less than p_w according to Ch.3 Sec.3.

Load on the deckhouse bottom:

— the load on the deckhouse bottom is governed by the load acting on the particular deck on which the deckhouse is located. Additionally, the support forces resulting from the weight induced loads shall be taken into account.

Load on deck beams and girders:

— for designing the deck beams and girders of the deck on which the deckhouse is located the following loads shall be taken:
    — below the deckhouse: load p_u according to the pressure head due to the distance between the supporting deck and the deckhouse bottom [kPa]
    — outside the deckhouse: load p_S
    — bearing forces in accordance with the weight induced load assumptions.

3.6.3 Load cases

For design purposes the service load cases and extraordinary load cases shall be investigated separately.

Service load cases:

— forces due to external loads acting in longitudinal, transverse and vertical direction shall be taken into account
— for designing the securing devices to prevent the deckhouse from being lifted, the force (in upward direction) shall not be taken less than 0.5 g G.

Extraordinary load cases:

— forces due to collision in longitudinal direction and forces due to static heel of 45° in transverse and vertical direction shall be taken into account
— the possible consequences of a fire for the elastic mounting of the deckhouse shall be examined (e.g. failure of rubber elastic mounting elements, melting of glue). Even in this case, the mounting elements between hull and deckhouse bottom shall be capable of withstanding the horizontal force according to extraordinary loads in transverse direction
— for designing of the securing devices to prevent the deckhouse from being lifted, a force not less than the buoyancy force of the deckhouse resulting from a water level of 2 m above the freeboard deck shall be taken.

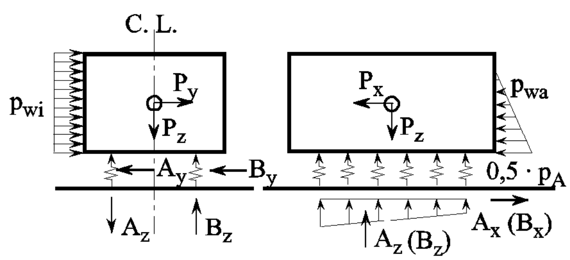


Figure 6 Design loads due to wind and water pressure

3.7 Scantlings of rails, mounting elements and substructures

3.7.1 General

The scantlings of those elements shall be determined in accordance with the service load cases. The effect of deflection of main girders need not be considered under the condition that the deflection is so negligible that all elements take over the loads equally.

Strength calculations for the structural elements with information regarding acting forces shall be submitted for approval.

3.7.2 Permissible stresses

The permissible stresses given in Table 1 shall not be exceeded in the rails and the steel structures of mounting elements and in the substructures (deck beams, girders of the deckhouse and the deck, on which the deckhouse is located).

The permissible stresses for designing the elastic mounting elements of various systems will be considered from case to case. Sufficient data shall be submitted for approval.

The stresses in the securing devices to prevent the deckhouse from being lifted shall not exceed the stress values specified in Table 1.

Table 1 Permissible stress in the rails and the steel structures at mounting elements and in the substructures [MPa]

Type of stress	*Service load cases*	*Extraordinary load cases*
Normal stress σ_n	$0.6 \cdot R_y \cdot f_m$	$0.75 \cdot R_y \cdot f_m$
Shear stress τ	$0.35 \cdot R_y \cdot f_m$	$0.43 \cdot R_y \cdot f_m$
Equivalent stress: $\sigma_v = \sqrt{\sigma_n^2 + 3 \cdot \tau^2}$	$0.75 \cdot R_y \cdot f_m$	$0.9 \cdot R_y \cdot f_m$

In screwed connections, the permissible stresses given in Table 2 shall not be exceeded.

Where turnbuckles in accordance with DIN 82008 are used for securing devices, the load per bolt under load conditions, (see [3.6.3]) may be equal to the proof load (two (2) times safe working load).

Table 2 Permissible stresses in screwed connections [MPa]

Type of stress	*Service load cases*	*Extraordinary load cases*
Longitudinal tension σ_n	$0.5 \cdot R_y$	$0.8 \cdot R_y$
Bearing pressure p_ℓ	$1.0 \cdot R_y$	$1.0 \cdot R_y$
Equivalent stress from longitudinal tension σ_n, tension τ_t, due to tightening torque and shear τ, if applicable: $\sigma_v = \sqrt{\sigma_n^2 + 3 \cdot \left(\tau^2 + \tau_t^2\right)}$	$0.6 \cdot R_y$	$1.0 \cdot R_y$

3.7.3 Corrosion addition

For the deck plating below elastically mounted deckhouses a minimum corrosion addition of t_c = 1.0 mm applies.

3.8 Helicopter decks

3.8.1 General

The starting/landing zone shall be designed for the largest helicopter type expected to use the helicopter deck. For helicopter decks forming a part of the hull girder the requirements of [3.1] to [3.5] shall be considered too.

The marking of the helicopter deck shall be done according to international or national regulations and standards.

3.8.2 Design loads

The loads as defined in Ch.3 Sec.3 [3.4.3] shall be considered separately.

3.8.3 Plating

The thickness of the plating shall be determined according to Sec.3 [5].

Proof of sufficient buckling strength shall be carried out in accordance with App.B for structural elements subjected to compressive stresses.

3.8.4 Secondary stiffeners

Bending moments and shear forces shall be calculated for the most unfavourable position of the helicopter with one or two loads F_E, whichever is possible.

3.8.5 Primary members

Primary members shall be calculated like secondary stiffeners [3.8.4].

If pillars would be part of the support of the helicopter deck the deadweight of the deck and the acceleration components a_x, a_y and a_z as well as the wind load on the deck structure have to be considered for dimensioning.

γ_m = 2.0 for normal landing

= 1.2 for crash/ emergency landing.

4 Watertight and non-watertight bulkheads

4.1 General

4.1.1 Watertight subdivision

The watertight subdivision will be determined in general by the damage stability calculations according to Ch.10.

Number and location of transverse bulkheads fitted in addition to those specified in Ch.2 Sec.1 shall be selected to ensure sufficient transverse strength of the hull.

4.1.2 Watertight trunks, tunnels, duct keels and ventilators shall be of the same strength as watertight bulkheads at corresponding levels.

4.2 Scantlings of single plate bulkheads

4.2.1 Design references

The requirements of this article apply to longitudinal and transverse bulkheads, which may be plane or corrugated.

The scantlings of plating, secondary and primary members of the bulkheads shall be determined according to Sec.3 [5] and [4].

For inclined bulkheads with an inclination to the horizontal of less than or equal 45° the requirements according to [3] (deck) shall be fulfilled.

Where spaces are intended to be used as tanks, their bulkheads and walls are also to comply with the requirements of [2].

4.2.2 Plating

Stern tube bulkheads shall be provided with a strengthened plate in way of the stern tube.

4.2.3 Secondary stiffeners

In general, end connections of stiffeners shall be bracketed. If bracketed end connections cannot be applied due to hull lines, other arrangements may be accepted. The end attachment of secondary stiffeners shall comply with App.A [8.3.2].

Unbracketed bulkhead stiffeners shall be connected to the decks by welding. The length of weld shall be at least 0.6 × depth of the section.

4.2.4 Primary members

The effective width shall be determined according to App.A [7.2].

Frames shall be connected to transverse deck beams by brackets according to App.A [8.4.4].

The transverse structure of superstructures and deckhouses shall be sufficiently dimensioned for stiffness by a suitable arrangement of end bulkheads, web frames, steel walls of cabins and casings, or by other measures.

In general, primary supporting members shall be dimensioned considering the elastic strength assessment. In this case $\gamma = 1.1$ shall be considered.

For the design of girders and web frames of bulkheads plastic hinges can be taken into account, if Sec.3 [6.3] is fulfilled. This can be done either by a non-linear calculation of the total bulkhead or by a linear girder grillage calculation of the idealized bulkhead.

4.3 Corrugated bulkheads

In addition to [4.2] following requirements shall be considered for corrugated bulkheads:

4.3.1 Plating

The greater one of the values *b* or *s* [m] according to [4.2] shall be taken into account for the spacing a.

4.3.2 Section modulus

The required section modulus of a corrugated bulkhead element shall be determined by direct calculation. For the spacing *a*, the width of an element *e*, in m, according to Figure 7 shall be taken. For the end attachment see App.A [8.3.2].

The actual section modulus of a corrugated bulkhead element shall be assessed according to the following equation (refer also to Figure 7):

$$W = t \cdot d \cdot \left(b + \frac{s}{3}\right) \ [\mathrm{cm}^3]$$

For flanges of corrugated elements subject to compressive stresses the effective width according to App.B [2.2] has to be considered.

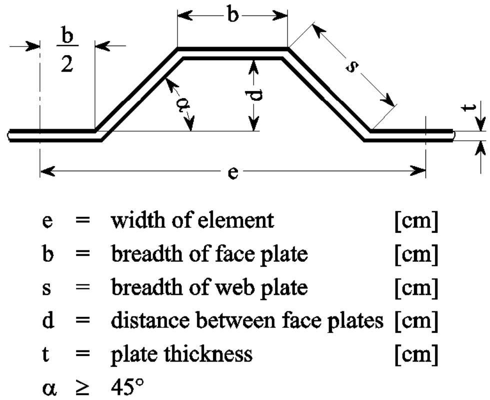


Figure 7 Dimensions of a corrugated bulkhead element

Care shall be taken that the forces acting at the supports of corrugated bulkheads are properly transmitted into the adjacent structure by fitting structural elements such as carlings, girders or floors in line with the corrugations.

Note:

Where carlings or similar elements cannot be fitted in line with the web strips of corrugated bulkhead elements, these web strips cannot be included into the section modulus at the support point for transmitting the moment of constraint.

Other than in given in [4.3.2] the section modulus of a corrugated element is then to be determined by the following equation:

$$W_{el} = t \cdot b \cdot (d + t) \text{ [cm}^3\text{]}$$

---e-n-d---o-f---n-o-t-e---

Where corrugated bulkheads are cut in way of primary supporting members, corrugations on each side of the primary member shall be aligned with each other.

The strength continuity of corrugated bulkheads shall be maintained at the ends of corrugations.

Where a bulkhead is provided with a lower stool, floors or girders shall be fitted in line with both sides of the lower stool. Where a bulkhead is not provided with a lower stool, floors or girders shall be fitted in line with both flanges of the vertically corrugated transverse bulkhead.

The supporting floors or girders shall be connected to each other by suitably designed shear plates.

At deck, if no upper stool is fitted, transverse or longitudinal stiffeners shall be fitted in line with the corrugation flanges.

When the corrugation flange connected to the adjoining boundary structures (i.e. inner hull, side shell, longitudinal bulkhead, trunk, etc) is smaller than 50% of the width of the typical corrugation flange, an advanced analysis of the connection is required.

Stool side plating shall be aligned with the corrugation flanges.

4.4 Shaft tunnels

4.4.1 General

Shaft and stuffing box shall be accessible. Where one or more compartments are situated between stern tube bulkhead and engine room, a watertight shaft tunnel shall be arranged for ships with one or three propellers. The size of the shaft tunnel shall be adequate for service and maintenance purposes.

The access opening between engine room and shaft tunnel shall be closed by a watertight sliding door complying with the requirements according to Ch.2 Sec.1 [3.2.3]. For extremely short shaft tunnels watertight doors between tunnel and engine room may be dispensed with subject to special approval.

4.5 Non-tight bulkheads

In general, openings in wash bulkheads shall have generous radii and their aggregate area shall not be less than 10% of the area of the bulkhead.

APPENDIX A GENERAL DESIGN PRINCIPLES

For symbols not defined in this section, see Ch.1 Sec.4.

1 General

1.1 Introduction

1.1.1 The requirements regarding the application of various structural materials as well as protection methods are given in [1] to [6]. The requirements and definition of structural idealisation and structural detail principles are given in [7] and [8].

1.1.2 The requirements to manufacture, condition of supply, heat-treatment, testing, inspection, tolerances, chemical composition, mechanical properties, repair, identification, certification etc. shall in general follow the relevant chapter of Pt.2.

1.1.3 In Table 1 the usually applied steels for hull, superstructures and deckhouses are summarized.

1.1.4 The usually applied aluminium alloys are defined in Table 4.

1.2 Material certificates

1.2.1 Rolled steel and aluminium for hull structures are normally to be supplied with the Society's material certificates in compliance with the requirements given in DNVGL-RU-SHIP Pt.2.

1.2.2 Requirements for material certificates for forgings, castings and other materials for special parts and equipment are stated in connection with the rule requirements for each individual part.

2 Rolled steels for structural application

2.1 General

2.1.1 Where the subsequent rules for material grade are dependent on plate thickness, the requirements are based on the thickness as built. For vessels with L < 90 m, where the applied plate thickness is greater than that required by the rules, a lower material grade may be applied, after special consideration.

Guidance note:

Attention should be drawn to the fact when the hull plating is being gauged at periodical surveys and the wastage considered in relation to reductions allowed by the Society, such allowed reductions are based on the nominal thicknesses required by the rules.

The under thickness tolerances acceptable for classification should be seen as the lower limit of a total minus-plus standard range of tolerances which could be met in normal production with a conventional rolling mill settled to produce in average the nominal thickness.

However, with modern rolling mills it might be possible to produce plates to a narrow band of thickness tolerances which could permit to consistently produce material thinner than the nominal thickness, satisfying at the same time the under thickness tolerance given in Pt.2.

Therefore in such a case the material will reach earlier the minimum thickness allowable at the hull gaugings.

It is upon the shipyard and owner, bearing in mind the above situation, to decide whether, for commercial reasons, stricter under thickness tolerances should be specified in the individual cases.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

2.1.2 Young's modulus and Poisson's ratio

The Young's modulus for carbon steel materials is equal to 206 GPa and the Poisson's ratio equal to 0.3.

2.1.3 Steel material grades and mechanical properties

Full details for requirements to materials are given in Pt.2 Ch.2.

Steel having a specified minimum yield stress of 235 MPa is regarded as normal strength hull structural steel and is denoted by MS for mild steel. Steel having a higher specified minimum yield stress is regarded as higher strength hull structural steel and is denoted HT for high tensile steel.

Where structural members are completely or partly made from higher strength hull structural steel, a suitable notation will be entered into the ship's certificate.

Material grades of hull structural steels are referred to as follows:

a) A, B, D and E denote normal strength steel grades.

b) AH, DH and EH denote higher strength steel grades.

Guidance note:

Especially when higher strength hull structural steels are used, limitation of permissible stresses due to buckling and fatigue strength criteria may be required.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

Table 1 gives specified yield stress and tensile strength for selected steels generally used in construction of ships.

Table 1 Strength properties of selected steel materials

Material	*Material number* [1)]	*E* *[GPa]*	*R_y or $R_{p0.2}$* *[MPa]*	*R_m* *[MPa]*
Normal strength hull structural steel				
VL-A	1.0440	206	235	400 - 520
VL-B	1.0442			
VL-D	1.0474			
VL-E	1.0476			
Higher strength hull structural steel				
VL-A 32	1.0513	206	315	440
VL-D 32	1.0514			
VL-E 32	1.0515			
VL-A 36	1.0583	206	355	490
VL-D 36	1.0584			
VL-E 36	1.0589			
VL-A 40	1.0532	206	390	510
VL-D 40	1.0534			
VL-E 40	1.0560			
Austenitic steel				
X2CrNiMnMoN Nb21-16-5-3	1.3964	195	430	700

Material	*Material number* [1)]	*E* *[GPa]*	*R_y or $R_{p0.2}$* *[MPa]*	*R_m* *[MPa]*
X2CrNiMnMoN Nb23-17-6-3	1.3974		510	800
1) defined in Key of Steels, Verlag Stahlschlüssel Wegst GmbH, D-71672 Marbach/Neckar				

2.1.4 Onboard documents

It is required to keep onboard a plan indicating the steel types and grades adopted for the hull structures. Where steels other than those indicated in Table 1 are used, their mechanical and chemical properties, as well as any workmanship requirements or recommendations, shall be available onboard together with the above plan.

2.2 Steel grades

Materials in the various strength members shall not be of lower grade than those corresponding to the material classes and grades specified in Table 2. General requirements are given in Table 2. The material grade requirements for hull members of each class depending on the thickness are defined in Table 3.

Table 2 Material classes and grades

Structural member category		*Material class/grade*
Secondary	A1. Longitudinal bulkhead strakes, other than those belonging to the primary category A2. Deck plating exposed to weather, other than that belonging to the primary or special category A3. Side plating	— Class I within 0.4 *L* amidships — Grade A/AH outside 0.4 *L* amidships
Primary	B1. Bottom plating, including keel plate B2. Strength deck plating, excluding that belonging to the special category B3. Continuous longitudinal plating of strength members above strength deck, B4. Uppermost strake in longitudinal bulkhead	— Class II within 0.4 *L* amidships — Grade A/AH outside 0.4 *L* amidships

Structural member category		Material class/grade
Special	C1. Sheer strake at strength deck [1] C2. Stringer plate in strength deck [1] C3. Deck strake at longitudinal bulkhead	— Class III within 0.4 *L* amidships — Class II outside 0.4 *L* amidships — Class I outside 0.6 *L* amidships
	C4. Strength deck plating at outboard corners of large openings	— Class III within 0.4 *L* amidships — Class II outside 0.4 *L* amidships — Class I outside 0.6 *L* amidships — Min. Class III within 0.4 *L*
	C5. Strength deck plating at corners of cargo hatch openings in bulk carriers, ore carriers combination carriers and other ships with similar hatch opening configurations	— Class III within 0.6 *L* amidships — Class II within rest of 0.4 *L*
	C6. Bilge strake of ships with double bottom over the full breadth and with length less than 150 m	— Class II within 0.6 *L* amidships — Class I outside 0.6 *L* amidships
	C7. Bilge strake in other ships [1]	— Class III within 0.4 *L* amidships — Class II outside 0.4 *L* amidships — Class I outside 0.6 *L* amidships
1) Single strakes required to be of class III within 0.4 *L* amidships shall have breadths not less than 800 + 5 *L*, in mm, need not be greater than 1800 mm, unless limited by the geometry of the ship's design.		

Table 3 Material grade requirements for classes I, II and III

Class	*I*		*II*		*III*	
As-built thickness, in mm	*MS*	*HT*	*MS*	*HT*	*MS*	*HT*
$t \le 15$	A	AH	A	AH	A	AH
$15 < t \le 20$	A	AH	A	AH	B	AH
$20 < t \le 25$	A	AH	B	AH	D	DH
$25 < t \le 30$	A	AH	D	DH	D	DH
$30 < t \le 35$	B	AH	D	DH	E	EH
$35 < t \le 40$	B	AH	D	DH	E	EH
$40 < t \le 150$	D	DH	E	EH	E	EH

Plating materials for stern frames and shaft brackets are in general not to be of lower grades than corresponding to class II.

2.3 Through thickness property

2.3.1 Where tee or cruciform connections employ partial or full penetration welds, and the plate material is subject to significant tensile strain in a direction perpendicular to the rolled surfaces, consideration shall be given to the use of special material with specified through thickness properties, in accordance with DNVGL-RU-SHIP Pt.2. These steels shall be designated on the approved plan by the required steel strength grade followed by the letter Z (e.g. EH36Z).

2.4 Structures exposed to low temperature

2.4.1 The material selection for structural members which are continuously exposed to temperatures below 0º C will be specially considered. The requirements are given in DNVGL-RU-SHIP Pt.6 Ch.6.

2.5 Austenitic steel

2.5.1 Where austenitic steels are applied having a ratio $R_{p0.2}/R_m \leq 0.5$, after special approval the 1.0% nominal proof stress $R_{p1.0}$ may be used for scantling purposes instead of the 0.2% proof stress $R_{p0.2}$.

2.6 Cold formed plating

2.6.1 For highly stressed components of the hull girder where notch toughness is of particular concern (e.g. items required to be class III in Table 2, such as radius gunwales and bilge strakes) the inside bending radius, in cold formed plating, shall not be less than 10 times the gross plate thickness for carbon-manganese steels (hull structural steels). The allowable inside bending radius may be reduced below 10 times the gross plate thickness, providing the additional requirements stated in [2.6.3] are complied with.

2.6.2 For important structural members not covered by [2.6.1] e.g. corrugated bulkheads, the inside bending radius in cold formed plating shall not be less than 4.5 times the plate thickness for carbon-manganese steels and two (2) times the plate thickness for austenitic- and ferritic-austenitic (duplex) stainless steels, corresponding to 10% and 20% theoretical deformation, respectively.

2.6.3 For carbon-manganese steels the allowable inside bending radius may be reduced below 4.5 times the plate thickness providing the following additional requirements are complied with:

a) The steel is killed and fine grain treated, i.e. grade D/DH or higher.
b) The material is impact tested in the strain-aged condition and satisfies the requirements stated herein. The deformation shall be equal to the maximum deformation to be applied during production, calculated by the equation

$$\frac{t_{grs}}{\left(2 \cdot r_{bdg} + t_{grs}\right)},$$

where t_{grs} is the gross thickness of the plate material and r_{bdg} is the bending radius. One sample shall be plastically strained at the calculated deformation or 5%, whichever is greater and then artificially aged at 250°C for one hour then subject to Charpy V-notch testing. The average impact energy after strain ageing shall meet the impact requirements specified for the grade of steel used.
c) 100% visual inspection of the deformed area shall be carried out. In addition, random check by magnetic particle testing shall be carried out.
d) The bending radius is in no case to be less than two (2) times the plate thickness.

3 Steel castings and forgings for structural application

3.1 General

3.1.1 The requirements to manufacture, condition of supply, heat-treatment, testing, inspection, tolerances, chemical composition, mechanical properties, repair, identification, certification etc. for steel castings and forging to be used for structural members shall in general follow the given requirements in DNVGL-RU-SHIP Pt.2.

3.2 Steel forgings for structural application

3.2.1 Rolled bars for structural application may be accepted in lieu of forged products, after consideration by the Society on a case-by-case basis. In such case, compliance with the applicable requirements of the rules for materials of the Society, relevant to the quality and testing of rolled parts accepted in lieu of forged parts, may be required.

3.3 Steel castings for structural application

3.3.1 Cast parts intended for stems and stern frames in general may be made of C and C-Mn weldable steels, having specified minimum tensile strength, R_m = 400 MPa, in accordance with the applicable requirements of the Society's rules for materials.

3.3.2 The welding of cast parts to main plating contributing to hull strength members is considered by the Society on a case-by-case basis.

The Society may require additional properties and tests for such casting, in particular impact properties which are appropriate to those of the steel plating on which the cast parts shall be welded and non-destructive examinations.

3.3.3 Heavily stressed cast parts of steering gear, particularly those intended to form a welded assembly and tillers or rotors mounted without key, additional surface and volumetric non-destructive examination to check their internal structure and integrity may be required by the Society.

4 Aluminium alloys

4.1 General

4.1.1 The strength properties for some typical aluminium alloys are given in Table 4 for unwelded and welded conditions. Other aluminium alloys may be considered provided that the specification (manufacture, chemical composition, temper, mechanical properties, welding, etc.) and the scope of application is submitted to the Society and approved.

4.1.2 For rolled products taking part in the longitudinal strength, alloys marked A shall be used. The alloy shall be chosen considering the stress level concerned.

4.1.3 In weld zones of rolled or extruded products (heat affected zones) the mechanical properties given for extruded products may in general be used as basis for the scantling requirements.

Note that for the series 6000 alloy the most unfavourable properties corresponding to -T4 condition shall be used.

4.1.4 Welding consumables giving a deposit weld metal with mechanical properties not less than those specified for the weld zones of the parent material shall be chosen.

In the case of structures subjected to low service temperatures or intended for other specific applications, the alloys to be employed shall be agreed case-by-case.

4.1.5 Unless otherwise agreed, the Young's modulus for aluminium alloys is equal to 70 GPa and the Poisson's ratio equal to 0.33.

4.2 Extruded plating

4.2.1 Extrusions with built-in plating and stiffeners, referred to as extruded plating, may be used.

4.2.2 In general, the application of extruded plating is limited to decks, bulkheads, superstructures and deckhouses. Other uses may be permitted by the Society on a case-by-case basis.

4.2.3 Extruded plating shall be oriented so that the stiffeners are parallel to the direction of main stresses.

4.2.4 Connections between extruded plating and primary members shall be given special attention.

4.3 Mechanical properties of weld joints

4.3.1 Welding heat input lowers locally the mechanical strength of aluminium alloys hardened by work hardening (series 5000 other than condition O or H111) or by heat treatment (series 6000).

4.3.2 Consequently, where necessary, a drop in mechanical characteristics of welded structures shall be considered in the heat-affected zone, with respect to the mechanical characteristics of the parent material.

4.3.3 The extension of the heat-affected zone depends on the type of welded joint, weld thickness etc. At least, an extent of not less than 25 mm on each side of the weld axis shall be taken into account.

4.3.4 Aluminium alloys of series 5000 in 0 condition (annealed) or in H111 condition (annealed flattened) are not subject to a drop in mechanical strength in the welded areas.

Aluminium alloys of series 5000 other than condition 0 or H111 are subjected to a drop in mechanical strength in the welded areas. The mechanical characteristics to consider in welded condition are, normally, those of condition 0 or H111, except otherwise indicated in Table 4. Higher mechanical characteristics may be taken into account, provided they are duly justified.

4.3.5 Aluminium alloys of series 6000 are subject to a drop in mechanical strength in the vicinity of the welded areas. The mechanical characteristics to be considered in welded condition are, normally, to be indicated by the supplier, if not indicated in Table 4.

4.3.6 For forgings and castings to be applied, requirements for chemical composition and mechanical properties shall be defined in each separate case by the Society.

4.3.7 In case of structures subjected to low service temperatures (i.e. below −25°C) or intended for other particular applications, the alloys to be employed shall be agreed in each separate case by the Society, who will state the acceptability requirements and conditions.

Table 4 Aluminium alloys for welded construction

<table>
<tr><th colspan="8">Guaranteed mechanical characteristics [1]</th></tr>
<tr><th colspan="4">Aluminium alloy</th><th colspan="2">Unwelded condition</th><th colspan="2">Welded condition</th></tr>
<tr><th>Alloy [2]</th><th>Products</th><th>Temper [2]</th><th>Thickness [mm]</th><th>$R_{p0.2}$ $[N/mm^2]$ [4]</th><th>R_m $[N/mm^2]$ [5]</th><th>$R_{p0.2}'$ $[N/mm^2]$ [4]</th><th>R_m' $[N/mm^2]$ [5]</th></tr>
<tr><td rowspan="3">5083</td><td rowspan="3">rolled</td><td>0 / H111 / H112</td><td rowspan="3">$t \leq 50$</td><td>125</td><td>275</td><td>125</td><td rowspan="3">275</td></tr>
<tr><td>H116</td><td rowspan="2">215</td><td rowspan="2">305</td><td rowspan="2">125</td></tr>
<tr><td>H32 / H321</td></tr>
</table>

Guaranteed mechanical characteristics [1]							
Aluminium alloy				Unwelded condition		Welded condition	
Alloy [2]	Products	Temper [2]	Thickness [mm]	$R_{p0.2}$ [N/mm²] [4]	R_m [N/mm²] [5]	$R_{p0.2}'$ [N/mm²] [4]	R_m' [N/mm²] [5]
5083	extruded	0 / H111	$t \leq 50$	110	270	110	270
		H112		125			
5086	rolled	0 / H111 / H112	$t \leq 50$	100	240	100	240
		H116		195	275		
		H32 / H321		185			
5086	extruded	0 / H111 / H112	$t \leq 50$	95	240	95	240
5383	rolled	0 / H111	$t \leq 40$	145	290	145	290
		H116 / H321		220	305		
5383	extruded	0 / H111	$t \leq 50$	145	290	145	290
		H 112		190	310		
5059	rolled	0 / H111	$t \leq 50$	160	330	155	300
		H116 / H321	$t \leq 20$	270	370		
			$20 < t \leq 40$	260	360		
5059	extruded	H 112	$t \leq 50$	200	330	155	300
5454	rolled	0 / H111	$t \leq 40$	85	215	85	215
		H32		180	250		
5754	rolled	0 / H111 / H112	$t \leq 50$	80	190	80	190
		H32	$t \leq 40$	165	240		
6005A	extruded	T5 / T6	$t \leq 50$	215	260	115	165
6060	extruded	T5	$t \leq 5$	120	160	65	95
			$5 < t \leq 25$	100	140		
6061	extruded	T5 / T6	$t \leq 50$	240	260	115	165
6082	extruded	T5 / T6	$t \leq 50$	260	310	115	170
6106	extruded	T6	$t \leq 10$	200	250	65	130

1) The guaranteed mechanical characteristics in this table correspond to general standard values. For more information, refer to the mini- mum values guaranteed by the product supplier. Higher values may be accepted on the basis of welding tests including recurrent workmanship test at the shipyard only.

2) Other grades or tempers may be considered, subject to the Society's agreement.

3) 6060 alloy shall not be for structural members sustaining impact loads (e.g. bottom longitudinals). The use of alloy 6106 is recommended in that case.

4) $R_{p0.2}$ and $R_{p0.2}'$ are the minimum guaranteed proof strengths at 0.2% in unwelded and welded condition respectively.

5) R_m and R_m' are the minimum guaranteed tensile strengths in unwelded and welded condition respectively.

4.4 Connection between steel and aluminium

4.4.1 Details of the proposed method of joining any aluminium and steel structures shall be submitted for approval.

4.4.2 To prevent galvanic corrosion a non-hygroscopic insulation material shall be applied between steel and aluminium when bolted connection.

4.4.3 Aluminium plating connected to steel boundary bar at deck is as far as possible to be arranged on the side exposed to moisture.

4.4.4 A rolled compound (aluminium/steel) bar may be used in a welded connection after special approval.

4.4.5 Direct contact between exposed wooden materials, e.g. deck planking, and aluminium shall be avoided.

4.4.6 Bolts with nuts and washers are either to be of stainless steel or cadmium plated or hot galvanized steel. The bolts shall be fitted with sleeves of insulating material. The spacing is normally not to exceed 4 times the bolt diameter.

4.4.7 For earthing of insulated aluminium superstructures, see Pt.4 Ch.8.

4.5 Others

4.5.1 Aluminium fittings in tanks used for the carriage of oil, and in cofferdams and pump rooms shall be avoided. Where fitted, aluminium fittings, units and supports, in tanks used for the carriage of oil, cofferdams and pump rooms shall satisfy the requirements of Pt.2 for aluminium anodes.

4.5.2 The underside of heavy portable aluminium structures such as gangways, shall be protected by means of a hard plastic or wood cover, or other approved means, in order to avoid the creation of smears. Such protection shall be permanently and securely attached to the structures.

5 Steel sandwich panel construction

5.1 Application

5.1.1 Steel sandwich panel construction for marine use may be applied in structural panels provided the strength of the sandwich construction is equivalent to that required in the Society's rules for a stiffened steel structure and the fire safety is equivalent to that required in the Society's rules for a steel construction.

Guidance note:

For SOLAS vessels fire safety equivalence of steel sandwich panel needs being explicitly demonstrated and documented by an analysis according to Ch.II-2 Reg.17 of SOLAS when the construction deviates from the prescriptive requirements of the same Ch.II-2.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

5.1.2 Verification of compliance with the Society's rules shall be demonstrated with the class guideline DNVGL-CG-0154 App.A.

5.2 Other materials and products

5.2.1 General

Other materials and products such as parts made of iron castings, where allowed, products made of copper and copper alloys, rivets, anchors, chain cables, cranes, masts, derrick posts, derricks, accessories and wire ropes shall comply with the applicable requirements of the rules for materials of the Society.

The use of plastics or other special materials not covered by these rules shall be considered by the Society on a case-by-case basis. In such cases, the requirements for the acceptance of the materials concerned shall be agreed by the Society.

5.2.2 Iron cast parts

As a rule, the use of grey iron, malleable iron or spheroidal graphite iron cast parts with combined ferritic/perlitic structure is allowed only to manufacture low stressed elements of secondary importance.

Ordinary iron cast parts may not be used for windows or sidescuttles, the use of high grade iron cast parts of a suitable type shall be considered by the Society on a case-by-case basis.

6 Reduction of the corrosion risk by special measures in design and construction

6.1 General

Yachts, systems and components shall be designed with the aim of ensuring optimum corrosion protection through the application of suitable structural measures.

Amongst others, the following measures have proven their worth in practice:

— points at which moisture tends to collect shall be avoided as far as possible
— the structural design shall enable good accessibility for activities of active and passive corrosion protection
— accumulations of condensed water in steel structural elements shall be avoided by providing sufficient venting possibilities
— the surface shall be as smooth as possible
— stiffeners, internal parts and piping, etc. shall be arranged in areas less at risk from corrosion
— possibilities of performing cleaning and pickling after welding to be provided, esp. with austenitic steels
— avoiding corrosion by impingement of drops by using baffle plates
— chain intermittent welds only permissible in zones which are heat-insulated and free of condensed water
— burrs and sharp edges shall be rounded off in order to improve coating
— hollow components which are not accessible shall be sealed off completely.

The corrosion additions t_c defined in Sec.3 [5.1] are based on this assumption.

6.2 Special requirements for stainless steels and stainless steel castings

6.2.1 Protective measures

Stainless steels and stainless steel castings exhibit a passive surface state in seawater, as is the case in several other media. Accordingly, coating of structures of these types of steel is only recommended under special conditions. Depending on the composition and grain structure, stainless steels are sensitive to local corrosion, such as pitting and crevice corrosion.

6.2.2 Cathodic protection

Cathodic corrosion protection may prevent or reduce pitting and crevice corrosion; in the case of crevice corrosion the effect is limited, depending on the crevice geometry.

Guidance note:

Uncoated stainless steels need not to be protected cathodically if they are suitable for withstanding the corrosion stress. Coated stainless steels shall be cathodically protected in the submerged zone.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

6.2.3 Design and workmanship

The following fundamental principles shall be observed:

— crevices shall be avoided as far as possible, if this is not feasible, the crevice shall be made as wide as possible
— flanges shall be made of materials with a greater corrosion resistance
— heat transmission paths should be avoided
— welds to be executed in technically competent manner
— weld joints to be post-treated in a technically competent manner
— coarse mechanical grinding is not permitted
— the surface should be smooth as possible
— only suitable processing tools shall be used e.g. 'stainless steel brush'.

6.3 Special requirements for aluminium alloys

For hull structures or components of zinc-free aluminium materials which are continuously submerged in seawater, cathodic protection with a protective potential of less than –0.55 V by sacrificial anodes is required.

6.4 Combination of materials

6.4.1 Preventive measures shall be taken to avoid contact corrosion associated with the combination of dissimilar metals with different potentials in an electrolyte solution, such as seawater.

6.4.2 In addition to selecting appropriate materials, steps such as suitable insulation, an effective coating and the application of cathodic protection can be taken in order to prevent contact corrosion.

Any direct contact between steel and aluminium alloy shall be avoided in wetted or immersed areas

6.4.3 Any heterogeneous joining system is subject to the Society's agreement.

The use of transition joints made of aluminium/steel-cladded plates or profiles is subject to the Society’s agreement.

Transition joints shall be type-approved.

Qualifications tests for welding procedures shall be carried out for each joint configuration.

A welding booklet giving preparations and various welding parameters for each type of assembly shall be submitted for review.

6.5 Fitting-out and berthing periods

6.5.1 For protection against corrosion arising from stray currents, such as those occurring due to inappropriate direct-current electrical supply to the naval ship for welding or mains lighting, as well as those arising from direct-current supplies to other facilities e.g. shore cranes and neighbouring ships, the provision of even additional cathodic protection by means of sacrificial anodes is not suitable.

6.5.2 Suitable measures shall be taken to prevent the formation of stray currents and suitable electric drainage shall be provided.

6.5.3 Particularly in the event of lengthy fitting-out periods, welding rectifiers shall be so arranged that stray currents can be eliminated. This is considered to be especially important for naval ships.

6.6 Corrosion protection

6.6.1 Structures to be protected

Dedicated seawater ballast tanks:

All dedicated seawater ballast tanks shall have an efficient corrosion prevention system.

Narrow spaces:

Narrow spaces are generally to be protected by an efficient protective product, particularly at the ends of the ship where inspections and maintenance are not easily practicable due to their inaccessibility.

6.6.2 Sacrificial anodes

All anodes shall be attached to the structure in such a way that they remain securely fastened both initially and during service even when it is wasted. The following methods are acceptable:

a) Steel core connected to the structure by continuous fillet welds.
b) Attachment to separate supports by bolting, provided a minimum of two bolts with lock nuts are used. However, other mechanical means of clamping may be accepted.

Anodes shall be attached to stiffeners or aligned in way of stiffeners on plane bulkhead plating, but they shall not be attached to the shell. The two ends shall not be attached to separate members which are capable of relative movement.

Where cores or supports are welded to local supporting members or primary supporting members, they shall be kept clear of end supports, toes of brackets and similar stress raisers. Where they are welded to asymmetrical members, the welding shall be at least 25 mm away from the edge of the web. In the case of stiffeners or girders with symmetrical face plates, the connection may be made to the web or to the centreline of the face plate, but well clear of the free edges.

6.6.3 Corrosion protection for ballast water tanks of passenger yachts

For the corrosion protection of ballast water tanks rules for coating of ballast water tanks are applicable. Protection by sacrificial anodes is not explicit required. *The IMO Performance Standard for Protective Coatings* (MSC 215(82)) shall be observed for vessel equal to or above 500 GT. Any agreed exemption from [4.3] shall observe the IACS UI SC 227.

7 Structural idealisation

Symbols

For symbols not defined in this section, see Ch.1 Sec.4.

7.1 Scantlings of stiffeners and girders

7.1.1 Definitions of primary and secondary stiffening members

Primary supporting members (girders) and secondary stiffening members (stiffeners) are defined as shown in Figure 1 and Figure 2.

The actual sectional properties shall be related to an axis parallel to the attached plating. The effective width of plating shall be calculated according to [7.2].

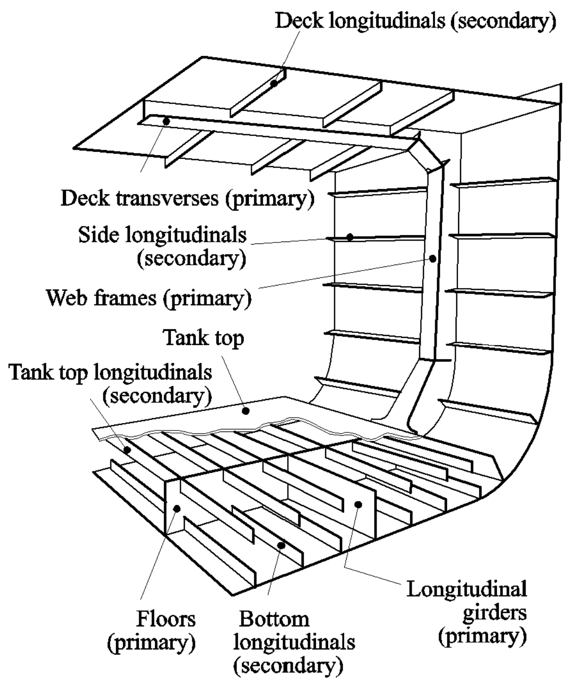


Figure 1 Definitions of primary supporting members and secondary stiffeners of the hull structure (longitudinal framing system)

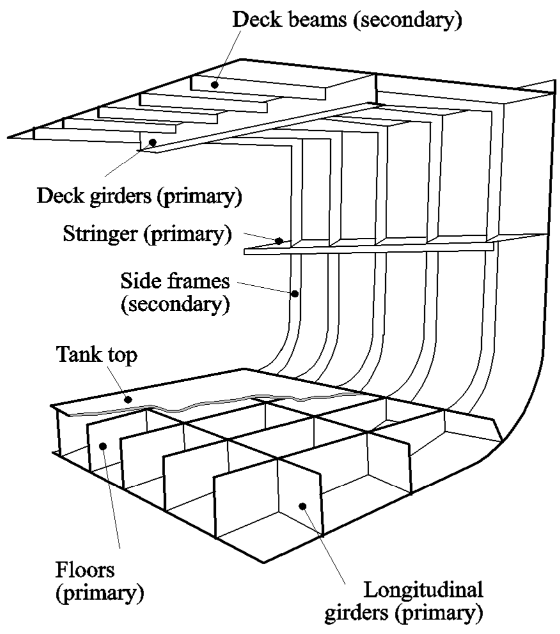


Figure 2 Definitions of primary supporting members and secondary stiffeners of the hull structure (transverse framing system)

7.2 Effective width of plating

7.2.1 Girders, frames and stiffeners

The effective width of plating e_m of stiffeners, frames and girders may be determined according to Table 5 considering the type of loading. Special calculations may be required for determining the effective width of one-sided or non-symmetrical flanges.

The effective area of plates shall not be less than the area of the face plate.

The effective width of stiffeners and girders subjected to compressive stresses may be determined according to App.B [2.2], but is in no case to be taken greater than determined by [7.2.1].

Table 5 Effective width of plating em of frames and girders

ℓ/e	0	1	2	3	4	5	6	7	≥ 8
e_{m1}/e	0	0.36	0.64	0.82	0.91	0.96	0.98	1.00	1.0
e_{m2}/e	0	0.20	0.37	0.52	0.65	0.75	0.84	0.89	0.9

e_{m1} shall be applied where girders are loaded by uniformly distributed loads or else by not less than six (6) equally spaced single loads

e_{m2} shall be applied where girders are loaded by three (3) or less single loads

Intermediate values may be obtained by direct interpolation.

ℓ = length between zero-points of bending moment curve, i.e. unsupported span in case of simply supported girders and 0.6 × unsupported span in case of constraint of both end of girders

e = width of plating supported, measured from centre of the adjacent unsupported fields.

7.2.2 Cantilevers

Where cantilevers are fitted at every frame, the effective width of plating may be taken as the frame spacing.

Where cantilevers are fitted at a greater spacing the effective width of plating at the respective cross section may approximately be taken as the distance of the cross section from the point on which the load is acting, however, not greater than the spacing of the cantilevers.

7.3 Unsupported span

7.3.1 Secondary stiffening members

The unsupported span is the true length of the stiffeners between two supporting girders or else their length including end attachments (brackets).

The frame spacings and spans are normally assumed to be measured in a vertical plane parallel to the centreline of the ship. However, if the ship's side deviates more than 10° from this plane, the frame distances and spans shall be measured along the side of the ship.

Instead of the true length of curved frames the length of chord between the supporting points can be selected.

7.3.2 Corrugated bulkhead elements

The unsupported span of corrugated bulkhead elements is their length between bottom or deck and their length between vertical or horizontal girders. Where corrugated bulkhead elements are connected to box type elements of comparatively low rigidity, their depth shall be included into the span unless otherwise proved by calculations.

7.3.3 Primary supporting members

The unsupported span of transverses and girders shall be determined according to Figure 3 depending on the type of end attachment.

In special cases, the rigidity of the adjoining girders shall be taken into account when determining the span of girder.

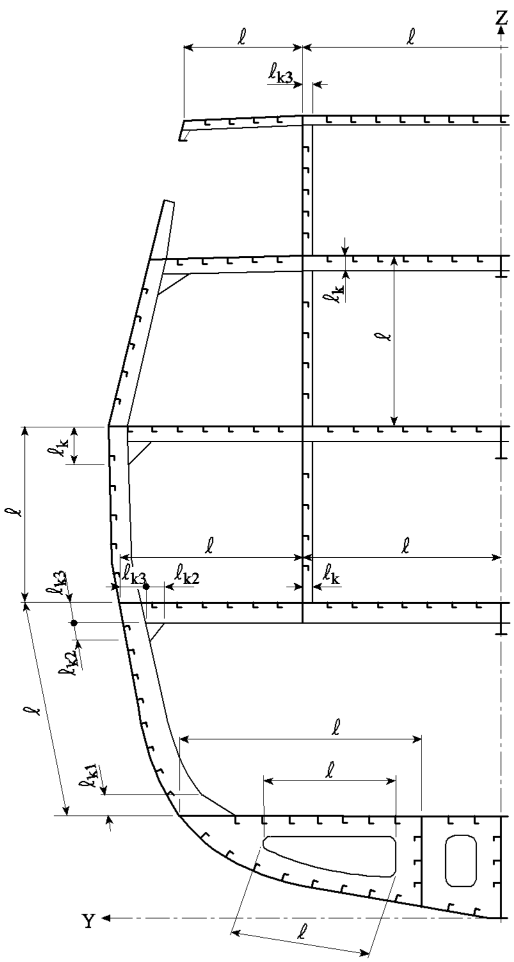


Figure 3 2D - transverse members

7.3.4 Geometrical properties of stiffeners and primary supporting members

Shear area of primary supporting members with web openings.

The effective web height, h_{eff}, in mm, to be considered for calculating the effective shear area shall be taken as the lesser of:

$h_{eff} = h_w$

$h_{eff} = h_{w3} + h_{w4}$

$h_{eff} = h_{w1} + h_{w2} + h_{w4}$

where:

h_w = web height, in mm

$h_{w1}, h_{w2}, h_{w3}, h_{w4}$ = dimensions as shown in Figure 4.

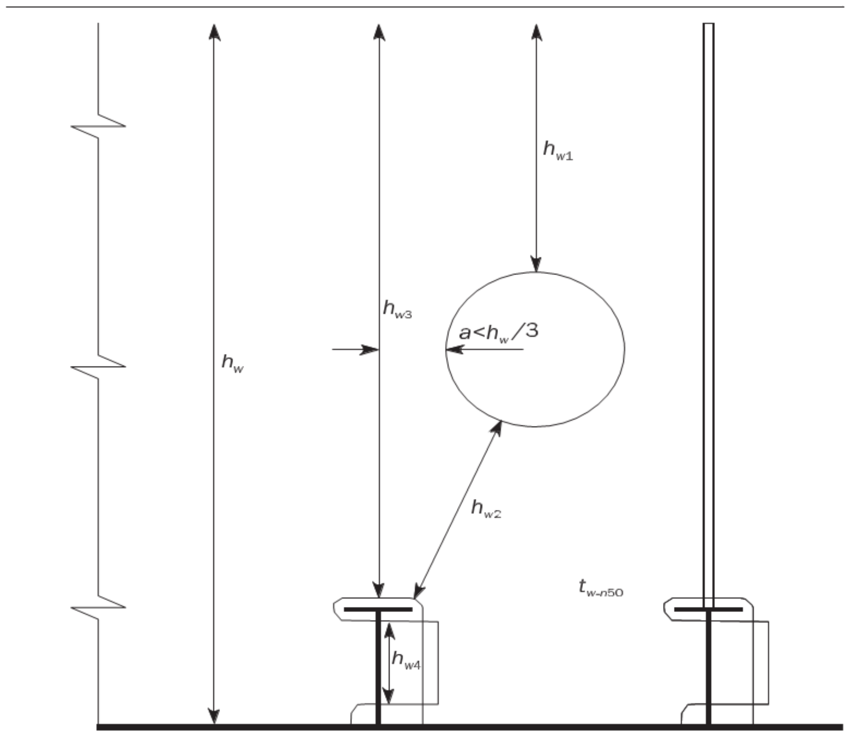


Figure 4 Effective shear area in way of web openings

Where the girder flange is not perpendicular to the considered cross section in the girder, the effective web area shall be taken as:

$$A_W = 0.01\, h_n\, t_w + 1.3\, A_{Fl} \sin 2\theta \sin \theta \ [\mathrm{cm}^2]$$

where:

h_n = as given in Figure 5
A_{Fl} = flange area in cm^2
θ = angle of slope of continuous flange
t_w = web thickness in mm.

See also Figure 5.

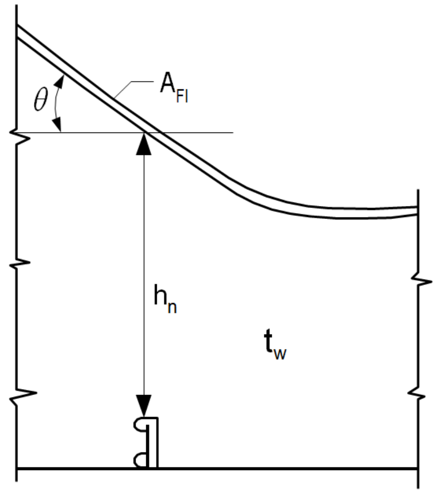


Figure 5 Effective web area in way of brackets

8 Structural detail principles

Symbols

For symbols not defined in this section, refer to Ch.1 Sec.4.

8.1 Application

8.1.1 General

If not specified otherwise, the requirements of this section apply to the hull structure, superstructures and deckhouses.

Alternative structural layout to what specified in this section may be considered based on direct calculations reflecting actual design.

8.1.2 Definitions

For determining scantlings of beams, stiffeners and girders the terms constraint and simple support will be used. Constraint will be assumed where for instance the stiffeners are rigidly connected to other members by means of brackets or are running throughout over supporting girders. Simple support will be assumed where for instance the stiffener ends are sniped or the stiffeners are connected to plating only.

8.2 General principles

8.2.1 Structural continuity

General:

Attention shall be paid to the structural continuity, in particular in the following areas:

— in way of changes in the framing system
— at end connections of primary supporting members or stiffeners
— in way of the transition zones between midship area and fore part, aft part and machinery space
— in way of side and end bulkheads of superstructures.

At the termination of a structural member, structural continuity shall be maintained by the fitting of suitable supporting structure.

Abrupt changes in transverse section properties of longitudinal members shall be avoided. Smooth transitions shall be provided.

Between the midship region and the end regions there shall be a gradual transition in plate thickness for bottom, shell and strength deck plating.

Longitudinal members:

Longitudinal members shall be arranged in such a way that continuity of strength is maintained.

Longitudinal members contributing to the hull girder longitudinal strength shall extend continuously as far as practicable within 0.8 L amidship. Structural continuity shall be ensured in way of end terminations. For longitudinal bulkheads or deep girders, large transition brackets (e.g. scarfing brackets) fitted in line with the longitudinal member are a possible means to achieve such structural continuity.

All longitudinal members taken into account for calculating the hull girder ultimate bending and shear capacity and the midship section modulus shall extend over the required length amidships and shall be tapered gradually to the required end scantlings, see also Sec.1.

Abrupt discontinuities of strength of longitudinal members shall be avoided as far as practicable. Where longitudinal members having different scantlings are connected with each other, smooth transitions shall be provided.

At the ends of longitudinal bulkheads or continuous longitudinal walls suitable shifting brackets shall be provided.

Flanged structural elements transmitting forces perpendicular to the knuckle, shall be adequately supported at their knuckle, i.e. the knuckles of the inner bottom shall be located above floors, longitudinal girders or bulkheads.

If longitudinal structures, such as longitudinal bulkheads or decks, include a knuckle which is formed by two butt-welded plates, the knuckle shall be supported in the vicinity of the joint rather than at the exact location of the joint. The minimum distance d to the supporting structure shall be at least.

$$d = 25 + \frac{t_f}{2}$$

but not more than 50 mm, see Figure 10. The increased stress peak due to distance *d* shall be considered and may be estimated according to [8.4.2].

Primary supporting members:

Primary supporting members shall be arranged in such a way that continuity of strength is maintained. Abrupt changes of web height or cross section shall be avoided.

Stiffeners:

Stiffeners shall be arranged in such a way that continuity of strength is maintained.

Stiffeners contributing to the hull girder longitudinal strength shall be continuous when crossing primary supporting members within the 0.4 *L* amidships and as far as practicable outside 0.4 *L* amidships.

Where stiffeners are terminated in way of large openings, foundations and partial girders, compensation shall be arranged to provide structural continuity in way of the end connection.

Sniped ends of stiffeners:

Stiffeners may be sniped at the ends, if no slamming occurs and if the thickness of the plating supported by stiffeners is not less than:

$$t = 15 \cdot \sqrt{\frac{F_s \cdot \gamma \cdot \gamma_m}{R_y}} + t_c \text{ [mm]}$$

where:

F_s = maximum support force [kN] to be transferred by the plating according to Ch.4 Table 3.4
= FA or FB (including load factors).

The corresponding load cases for the local design loads are given Ch.3 Sec.3.

Plating:

Where plates with different thicknesses are joined, a transition plate shall be added if the difference in the as-built plate thickness exceed 50% of larger plate thickness in the load carrying direction. This also applies to the strengthening by local inserts, e.g. insert plates in double bottom girders, floors and inner bottom.

Weld joints:

Weld joints shall be avoided in areas with high stress concentration.

8.2.2 Local reinforcements

Reinforcements at knuckles:

a) Knuckles are in general to be stiffened to achieve out-of-plane stiffness by fitting stiffeners or equivalent means in line with the knuckle.
b) Whenever a knuckle in a main member (shell, longitudinal bulkhead etc) is arranged, stiffening in the form of webs, brackets or profiles shall be connected to the members to which they shall transfer the load (in shear). See example of reinforcement at upper hopper knuckle in Figure 7.
c) Where stiffeners intersect the knuckle as shown in Figure 6, effective support shall be provided for the stiffeners in way of the knuckle, e.g. as indicated in Figure 6.
d) When the stiffeners of the shell, inner shell or bulkhead intersect a knuckle at a narrow angle, it may be accepted to interrupt the stiffener at the knuckle, provided that proper end support in terms of carling, bracket or equivalent is fitted. Alternative design solution with, e.g. closely spaced carlings fitted across the knuckle between longitudinal members above and below the knuckle is generally not recommended.
e) For longitudinal shallow knuckles, closely spaced carlings shall be fitted across the knuckle, between longitudinal members above and below the knuckle. Carlings or other types of reinforcement need not be fitted in way of shallow knuckles that are not subject to high lateral loads and/or high in-plane loads across the knuckle, such as deck camber knuckles.
f) Generally, the distance between the knuckle and the support stiffening in line with the knuckle shall not be greater than 50 mm. The increased stress peak in way of weld seam in the knuckle shall be considered. The stress concentration factor k_s may be estimated according to [8.4.2].

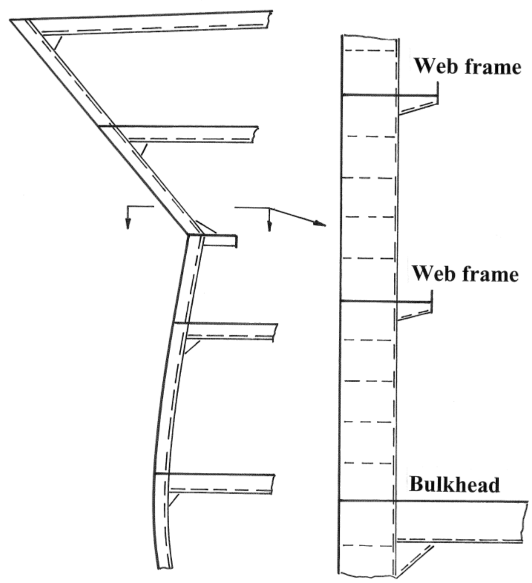


Figure 6 Reinforcement at knuckle

g) When a stiffener or primary supporting member is knuckled within the length of the span, effective support shall be provided by fitting tripping bracket or equivalent for the support of the face plate, and tripping bracket or equivalent for supporting the knuckled web section, see Figure 7.

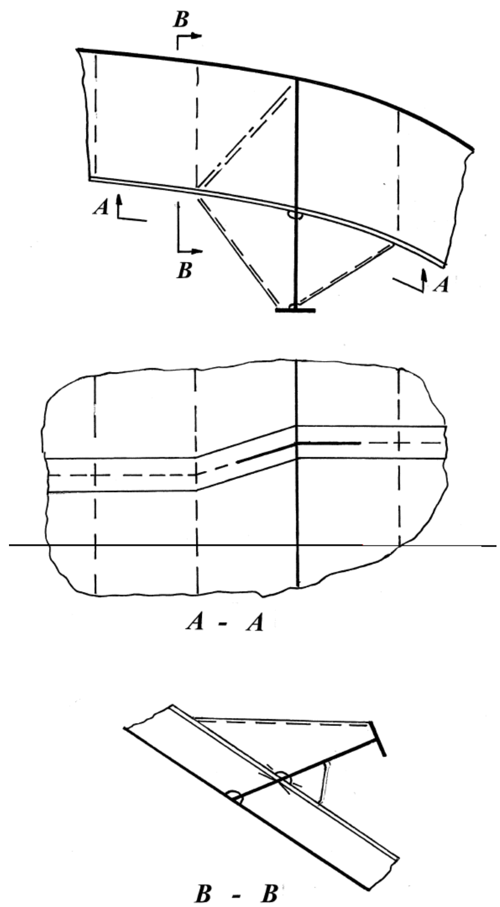


Figure 7 Support arrangement for knuckled stringer

8.2.2.1 Knuckle support at integral bracket:

If the flange transition between the stiffener and an integral bracket is knuckled, the flange shall be effectively supported in way of the knuckle. Alternatively the flange may be curved with radius according to [8.4.2].

Guidance note:

Shell stiffeners in the bow flare area, having an integral end bracket, are generally recommended to be tripping supported in way of the end bracket, also when the flange transition has been curved.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

Reinforcement in way of attachments for permanent means of access:

Local reinforcement, considering location and strength, shall be provided in way of attachments to the hull structure for permanent means of access.

Reinforcement of deck structure in way of concentrated loads:

The deck structure shall be reinforced in way of concentrated loads, such as anchor windlass, deck machinery, cranes, masts and derrick posts.

Reinforcement by insert plates:

Insert plates shall be made of materials with, at least, the same specified minimum yield stress and grade as the plates to which they are welded.

8.3 Stiffeners

8.3.1 General

All types of stiffeners (excluding web stiffeners) shall be connected at their ends. However, in special cases sniped ends may be permitted. Requirements for the various types of connections (bracketed, bracketless or sniped ends) are given in [8.3.2] to [8.3.4].

Where the angle between the web plate of the stiffener and the attached plating is less than 50 degree, a tripping bracket shall be fitted. If the angle between the web plate of an unsymmetrical stiffener and the attached plating is less than 50 degrees, the face plate of the stiffener shall be fitted on the open angle side, Figure 8.

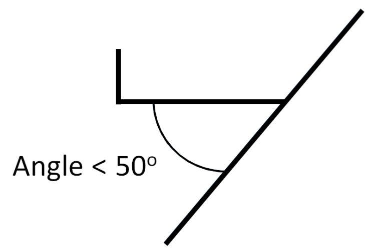


Figure 8 Angle between web plate and attached plating

8.3.2 Bracketed end connections of non-continuous stiffeners

Where continuity of strength of longitudinal members is provided by brackets, the alignment of the brackets on each side of the primary supporting member shall be ensured, and the scantlings of the brackets shall be such that the combined stiffener/bracket section modulus and effective cross sectional area are not less than those of the member.

Guidance note:

Note that end brackets for stiffeners may, as indicated in Figure 5, in general be arranged to be of the overlap type. End brackets of the type B, however, are only to be applied for locations where the bending moment capacity required for the bracket is reduced compared to the bending moment capacity of the stiffener, e.g. the upper end bracket of vertical stiffeners.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

The arrangement of the connection between the stiffener and the bracket shall be such that at no point in the connection the section modulus is less than that required for the stiffener.

8.3.3 Bracketless connections

Bracketless end connections may be applied for longitudinals and other stiffeners running continuously through girders (web frames, transverses, stringers, bulkheads etc.), provided sufficient connection area is arranged for.

Bracketed end connections are in general to be provided between non-continuous stiffeners on tight boundaries and continuous stiffeners on adjacent boundaries.

The design of bracketless connections shall be such as to provide adequate resistance to rotation and displacement of the connection.

Bracket toes and sniped stiffeners ends shall be terminated close to the adjacent member. The distance shall not exceed 40 mm unless the bracket or member is supported by another member on the opposite side of the plating. Tapering of the sniped end shall not be more than 30 deg. The depth of toe or sniped end is, generally, not to exceed the thickness of the bracket toe or sniped end member, but need not be less than 15 mm.

8.3.4 Stiffeners on watertight bulkheads

Bulkhead stiffeners cut in way of watertight doors shall be supported by carlings or stiffeners.

8.4 Primary supporting members (PSM)

8.4.1 General

Primary supporting members web stiffeners, tripping brackets and end brackets shall comply with [8.4.2] to [8.4.3]. Where the structural arrangement is such that these requirements cannot be complied with, adequate alternative arrangement has to be demonstrated by the designer.

8.4.2 Supporting members and cantilevers

In general the depth of girders should not be less than 1/25 of the unsupported span. The web depth of girders supporting continuous secondary stiffeners shall be at least 1.5 times the depth of the stiffeners.

Where transverses and girders fitted in the same plane are connected to each other, major discontinuities of strength shall be avoided. The web depth of the smaller girder shall, in general, not be less than 60% of the web depth of the greater one.

The taper between face plates with different dimensions shall be gradual. In general the taper shall not exceed 1 : 3. At intersections the forces acting in the face plates shall be properly transmitted.

For transmitting the acting forces the face plates shall be supported at their knuckles. For supporting the face plates of cantilevers, see Figure 9.

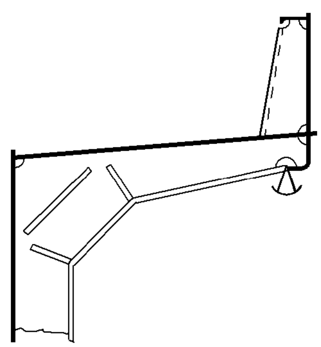


Figure 9 Support of the face plates of cantilevers

Upon special approval the stiffeners at the knuckles may be omitted if the following condition is complied with:

$$\sigma_a \leq \frac{\sigma_p \cdot b_e}{b_f} \text{ [MPa]}$$

where:

σ_a = actual stress in the face plate at the knuckle [MPa]
σ_p = permissible stress in the face plate [MPa]
b_f = breadth of face plate [mm]
b_e = effective breadth of face plate
= $t_w + n_1\left[t_f + c\left(b - t_f\right)\right]$ [mm]

t_W = web thickness [mm]

t_f = face plate thickness [mm]

b = $\frac{b_f - t_w}{n_1}$ [mm]

c = $\frac{1}{\frac{\left(b - t_f\right)^2}{R \cdot t_f} - n_2} + \frac{n_3 \cdot t_f}{\alpha^2 \cdot R}$

c_{max} = 1

2α = knuckle angle in [°], see Figure 10

α_{max} = 45°

R = radius of rounded face plates [mm]

= t_f for knuckled face plates

n_1 = 1 for unsymmetrical face plates (faceplate at one side only)

= 2 for symmetrical face plates

n_2 = 0 for face plates with one or two unsupported edges parallel to the web

= $\frac{\left(b - t_f\right)^2}{R \cdot t_f} \leq 1$

for face plates of multi-web girders

n_3 = 3 if no radial stiffener is fitted

= 3 000 if two or more radial stiffeners are fitted or if one knuckle stiffener is fitted according to Figure 10

$= \left(\frac{d}{t_f} - 8\right)^4$

if one stiffener is fitted according to Figure 10

$3 \leq n_3 \leq 3\ 000$

d = distance of the stiffener from the knuckle [mm].

For proof of fatigue strength of the weld seam in the knuckle, the stress concentration factor k_S (angle 2α according to Figure 10 < 35°) related to the stress σ_a in the face plate of thickness t_f may be estimated as follows and may be evaluated with case A5 of App.D Table 6:

$$k_s = \frac{t_f}{t_{f1}} \cdot \left[1 + \frac{6 \cdot n_4}{1 + \left(\frac{t_f}{t_{f1}}\right)^2} \cdot \tan\frac{2 \cdot \alpha \cdot t_{f1}}{R}\right]$$

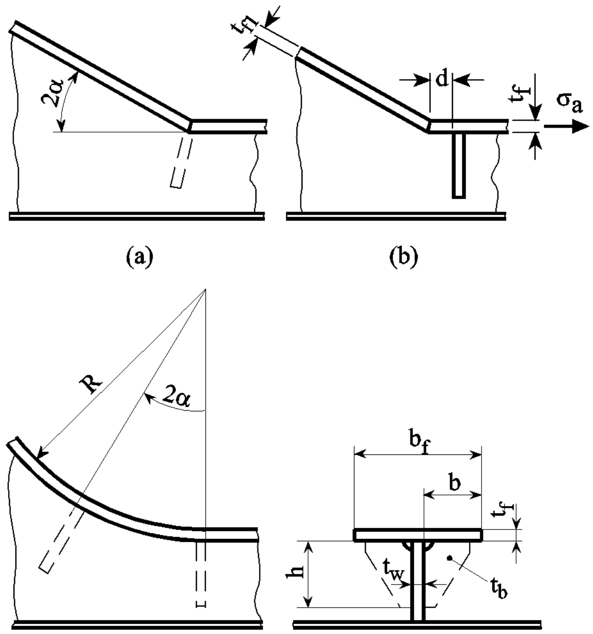


Figure 10 Location of stiffeners at knuckles

n_4 = 7.143 for $\frac{d}{t_f} > 8$

= $\frac{d}{t_f} - 0.51 \cdot \sqrt[4]{\frac{d}{t_f}}$ for $8 \geq \frac{d}{t_f} > 1.35$

= $0.5 \cdot \frac{d}{t_f} + 0.125$ for $1.35 \geq \frac{d}{t_f} \geq -0.25$

Scantlings of stiffeners (guidance):

thickness: t_b = $\frac{\sigma_a}{\sigma_p} t_f \cdot 2 \sin \alpha$

height: h = $1.5 \cdot b$

8.4.3 Tripping bracket arrangement

In general girders shall be provided with tripping brackets and web stiffeners to obtain adequate lateral and web panel stability. The requirements given below are providing for an acceptable standard. The stiffening system may, however, be modified based on direct stress analysis and stability calculations according to accepted methods.

Tripping brackets are generally to be fitted:

— at positions along the member span such that the spacing of these tripping elements shall not exceed 12 b_f, arranged alternately on both sides of the web for symmetrical face plates
— at the toe of end brackets
— at ends of continuous curved face plates
— in way of concentrated loads
— near the change of section

— in line with longitudinal stiffeners.

8.4.4 End connections

General:

Brackets or equivalent structure shall be provided at ends of primary supporting members. End brackets are generally to be soft-toed in areas considered critical with respect to fatigue.

Bracketless connections may be applied provided that there is adequate support of adjoining face plates.

Brackets:

For the scantlings of brackets the required section modulus of the stiffener is determining. Where stiffeners of different section moduli are connected to each other, the scantlings of the brackets are generally governed by the smaller stiffener

The thickness of brackets shall not be less than:

$$t' = c \cdot \sqrt[3]{M_{bp}}\ [\mathrm{mm}]$$

where:

c = 1.9 for non-flanged brackets
 = 1.5 for flanged brackets
M_{bp} = nominal bending moment [kNm] of profile with n = 1.0 but not less than the capacity of the profile M_U according to Sec.4 [4.2]
t_{min} = 3.0 mm
t_{max} = web thickness of smaller stiffener.

The arm length of brackets shall not be less than:

$$\ell = 1148 \cdot \frac{\sqrt[3]{M_{bp}}}{\sqrt{R_{yb}}} \cdot c_t\ [\mathrm{mm}]$$

where:

ℓ_{min} = 100 mm
M_{bp} = see Sec.4 [4.2]
R_{yb} = minimum yield strength of the bracket material [MPa]
C_t = $\sqrt{\frac{t}{t_a}}$
t_a = ''as built'' thickness of bracket [mm]
 ≥ t including t_c.

The arm length is the length of the welded connection.

Guidance note:

For deviating arm lengths the thickness of bracket shall be estimated by direct calculations considering sufficient safety against buckling.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

The throat thickness of the welded connection shall be determined according to Sec.3.

Where flanged brackets are used, the width of flange shall be determined according to the following equation:

$$b = 40 + 33 \cdot \frac{M_{bp}}{R_{yp}}$$

≤ 90 mm

M_{bp} = see previous description

R_{yp} = minimum yield strength of the profile material [MPa].

'Tween deck frames shall be connected to the main frames below. The end attachment may be carried out in accordance with Figure 11.

The following requirement shall be fulfilled:

$$M_{p0} \leq M_{p1} + M_{p2}$$

where:

M_{pi} = capacity of profile i (see Sec.4 [4.2]).

Index 0 refers to the profile with the greatest capacity of the connection point.

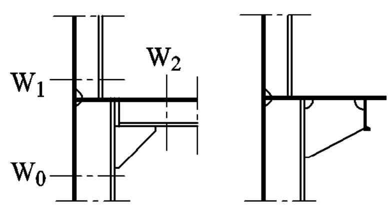


Figure 11 Connection of tween deck frames with decks and frames below

For a ring system where the end bracket is integral with the webs of the members and the face plate is carried continuously along the edges of the members and the bracket, the full area of the largest face plate shall be maintained close to the mid-point of the bracket and gradually tapered to the smaller face plates. Butts in face plates shall be kept well clear of the bracket toes.

Where a wide face plate abuts a narrower one, the taper shall not be greater than 1 : 4.

The toes of brackets shall not land on unstiffened plating. The toe height shall not be greater than the thickness of the bracket toe, but need not be less than 15 mm. In general, the end brackets of primary supporting members shall be soft-toed. Where primary supporting members are constructed of higher strength steel, particular attention shall be paid to the design of the end bracket toes in order to minimise stress concentrations.

Where a face plate is welded onto the edge or welded adjacent to the edge of the end bracket (see Figure 11), the face plate shall be sniped and tapered at an angle not greater than 30°.

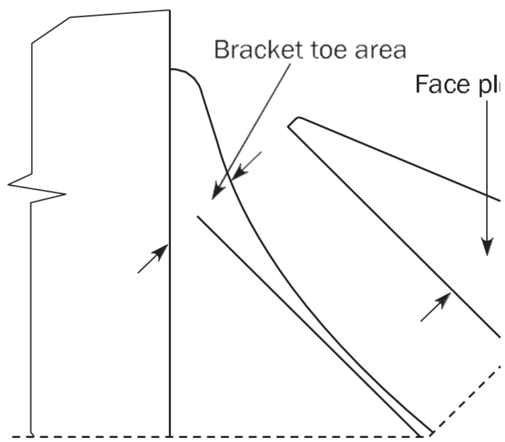


Figure 12 Bracket face plate adjacent to the edge

The details shown in this figure are only used to illustrate items described in the text and are not intended to represent design guidance or recommendations.

8.5 Openings

8.5.1 Openings and scallops in stiffeners

Figure 12 shows examples of air holes, drain holes and scallops. In general, the ratio of a/b as defined in Figure 12 shall be between 0.5 and 1.0, with a maximum b of 200 mm. In fatigue sensitive areas further consideration may be required with respect to the details and arrangements of openings and scallops.

Openings and scallops shall be kept at least 200 mm clear of the toes of end brackets, end connections and other areas of high stress concentration, measured along the length of the stiffener toward the mid-span and 50 mm measured along the length in the opposite direction, see Figure 13. In areas where the shear stress is less than 60 percent of the permissible stress, alternative arrangements may be accepted.

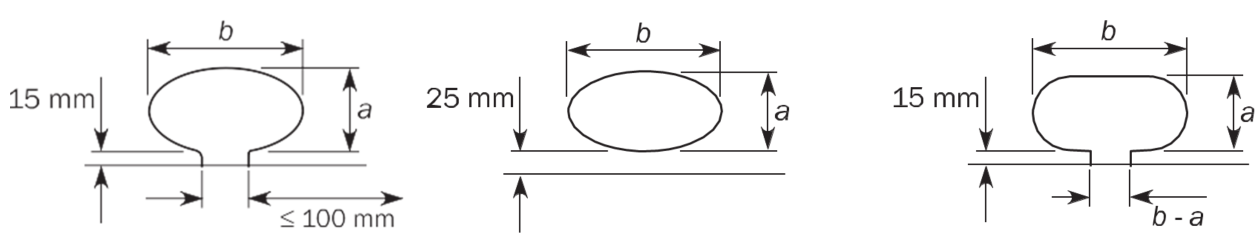


Figure 13 Examples of air holes, drain holes and scallops

The details shown in this figure are for guidance and illustration only.

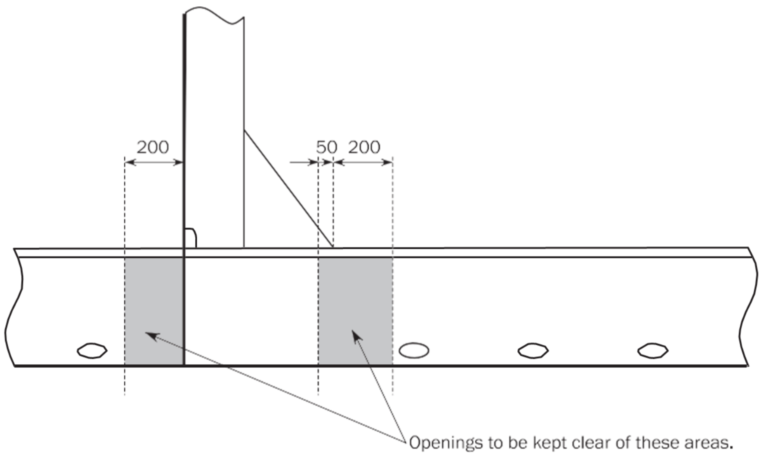


Figure 14 Location of air and drain holes

Closely spaced scallops or drain holes, i.e. where the distance between scallops/drain holes is less than twice the width *b* as shown in Figure 12, are not permitted in stiffeners contributing to the longitudinal strength. For other stiffeners, closely spaced scallops/drain holes are not permitted within 20% of the stiffener span measured from the end of the stiffener. Widely spaced air or drain holes may be permitted provided that they are of elliptical shape or equivalent to minimise stress concentration and are, in general, cut clear of the welds.

8.5.2 Openings in primary supporting members

General:

Manholes, lightening holes and other similar openings shall be avoided in way of concentrated loads and areas of high shear. In particular, manholes and similar openings shall be avoided in high stress areas unless the stresses in the plating and the panel buckling characteristics have been calculated and found satisfactory.

Examples of high stress areas include:

— vertical or horizontal diaphragm plates in narrow cofferdams/double plate bulkheads within one-sixth of their length from either end
— floors or double bottom girders close to their span ends
— primary supporting member webs in way of end bracket toes
— above the heads and below the heels of pillars.

Where openings are arranged, the shape of openings shall be such that the stress concentration remains within acceptable limits.

Openings shall be well rounded with smooth edges.

For preventing the face plates from tripping adequately spaced stiffeners or tripping brackets shall be provided. The spacing of these tripping elements shall not exceed 12 b_f, arranged alternately on both sides of the web for symmetrical face plates (see [8.4.4]).

The webs shall be stiffened to prevent buckling.

The location of lightening holes shall be such that the distance from hole edge to face plate is not less than 0.3 times the web depth.

In way of high shear stresses lightening holes in the webs shall be avoided as far as possible.

Openings in highly loaded structures should have the shorter dimension transverse to the direction of the main stresses. The corners of the plate have to be rounded-off and avoiding notch effects, grinding will be necessary in most cases.

Scallops, air- and drain holes:

Requirements are given in [8.5.1].

Manholes and lightening holes:

Web openings as indicated below do not require reinforcement:

— In single skin sections, having depth not exceeding 25% of the web depth and located so that the edges are not less than 40% of the web depth from the faceplate.
— In double skin sections, having depth not exceeding 50% of the web depth and located so that the edges are well clear of cut outs for the passage of stiffeners.

The length of openings shall not be greater than:

— At the mid-span of primary supporting members: the distance between adjacent openings.
— At the ends of the span: 25% of the distance between adjacent openings.

For openings cut in single skin sections, the length of opening shall not be greater than the web depth or 60% of the stiffener spacing, whichever is greater.

The ends of the openings shall be equidistant from the cut outs for stiffeners.

Where lightening holes are cut in the brackets, the distance from the circumference of the hole to the free flange of brackets shall not be less than the diameter of the lightening hole.

The diameter of the lightening holes in the bracket floors shall not be greater than 1/3 of the breadth of the brackets.

Openings not complying with this requirement shall be reinforced according to [8.5.2].

Reinforcements around openings:

Manholes and lightening holes shall be stiffened according to this requirement, except where alternative arrangements are demonstrated as satisfactory, in accordance with analysis methods.

On members contributing to longitudinal strength, stiffeners shall be fitted along the free edges of the openings parallel to the vertical and horizontal axis of the opening. Stiffeners may be omitted in one direction if the shortest axis is less than 400 mm and in both directions if length of both axes is less than 300 mm. Edge reinforcement may be used as an alternative to stiffeners, see Figure 14.

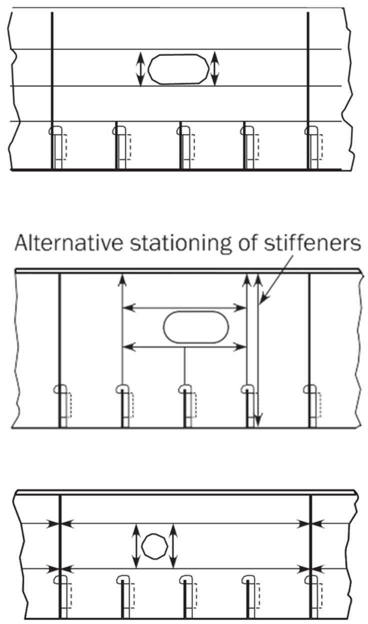


Figure 15 Web plate with openings

In the case of large openings in the web of PSMs (e.g. where a pipe tunnel is fitted in the double bottom), the secondary stresses in PSMs shall be considered for the reinforcement of these openings.

8.5.3 Evaluation of notch stress

Permissible notch stress:

The notch stress σ_K roughly calculated for linear-elastic material behaviour at free plate edges, e.g. at openings in decks, walls, girders etc., should, in general, fulfil the following criterion:

$$\sigma_k \leq f \cdot R_y$$

where:

f	=	1.1 for normal strength hull structural steel
	=	0.9 for higher strength hull structural steel with R_{eH} = 315 MPa
	=	0.8 for higher strength hull structural steel with R_{eH} = 355 MPa
	=	0.73 for higher strength hull structural steel with R_{eH} = 390 MPa.

For aluminium alloys the permissible notch stress has to be determined individually concerning the respective alloy.

If plate edges are free of notches and corners are rounded-off, a 20% higher notch stress σ_K may be permitted.

A further increase of stresses may be permitted on the basis of a fatigue strength analysis as described in App.D.

Notch factors to evaluate actual notch stress:

The actual notch stress can be determined by multiplying the nominal stress with the notch factor K_t. For some types of openings the notch factors are given in Figure 16 and Figure 17. An evaluation of notch stresses is possible by means of finite element calculations.

Guidance note:

These notch factors can only be used for girders with multiple openings if there is no correlation between the different openings regarding deformations and stresses.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

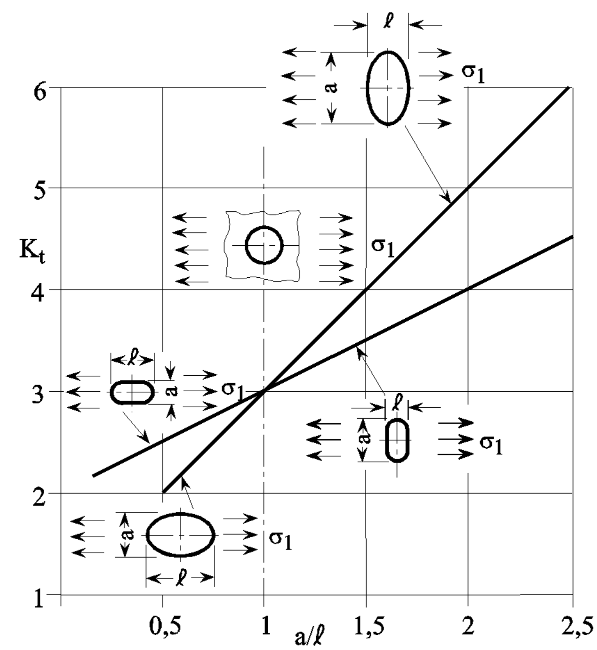


Figure 16 Notch factor K_t for rounded openings

Openings contributing to longitudinal strength:

All openings contributing to longitudinal strength shall have well rounded corners and a continuous longitudinal girder shall be fitted in way of the edges. Circular openings shall be edge-reinforced. The sectional area of the face bar shall not be less than:

$$A_f = 0.25 \cdot d \cdot t \text{ [cm}^2\text{]}$$

where:

d = diameter of openings [cm]

t = deck thickness [cm].

The reinforcing face bar may be dispensed with, where the diameter is less than 300 mm and the smallest distance from another opening is not less than 5 × diameter of the smaller opening. The distance between the outer edge of openings for pipes etc. and the ship's side shall not be less than the opening diameter.

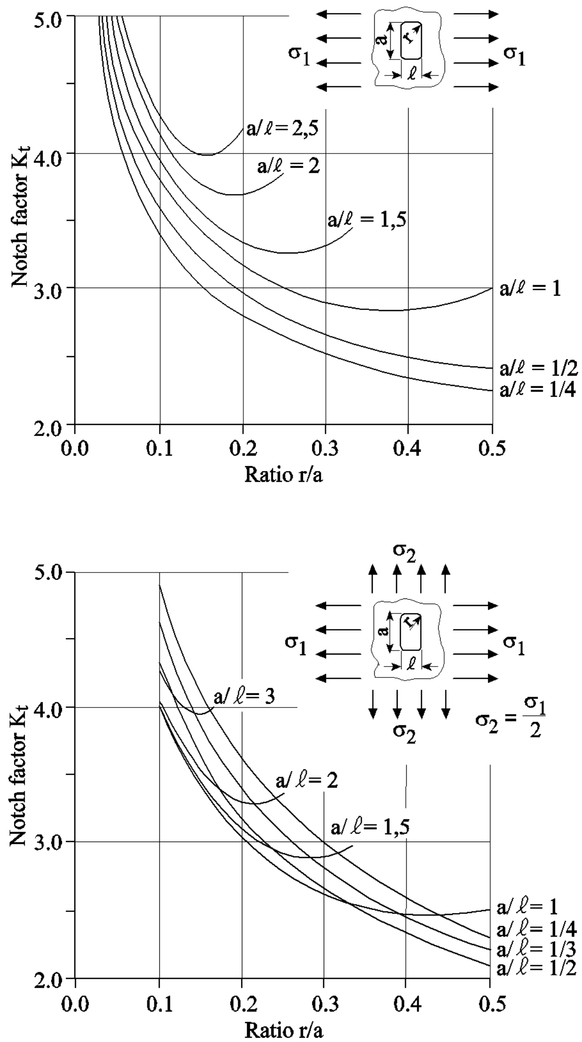


Figure 17 Notch factor K_t for rectangular openings with rounded corners at uniaxial state of stresses (above) and at biaxial state of stresses (below)

The corners of the opening shall be surrounded by strengthened plates which shall extend over at least one frame spacing fore-and-aft and athwart-ships. Within 0.5 *L* amidships, the thickness of the strengthened plate shall be equal to the deck thickness abreast the opening plus the deck thickness between the openings. Outside 0.5 *L* amidships the thickness of the strengthened plate need not exceed 1.6 times the thickness of the deck plating abreast the opening.

The corner radius shall not be less than:

$$r = n \cdot b \cdot \left(1 - \frac{b}{B}\right)$$

where:

r_{min} = 0.05 b, but not less than 0.1 m

n = $\frac{\ell}{200}$

n_{min} = 0.1

n_{max} = 0.25

ℓ = length of opening [m]

b = breadth or height [m], of the opening or total breadth of openings in case of more than one measured perpendicular to the longitudinal stresses.

b/B need not to be taken smaller than 0.4.

For side shell doors twice of the distance between centre of shell opening and the deck above may be taken into account for B in this equation.

Where the corners are elliptic or parabolic, strengthening according to previous paragraphs of this section is not required. The dimensions of the elliptical and parabolic corners shall be as shown in Figure 17. Where smaller values are taken for a and c, reinforced insert plates are required which will be considered in each individual case.

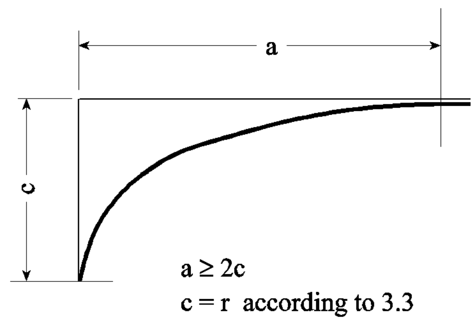


Figure 18 Design of elliptical or parabolic corners.

For ships with very large openings the design of the corner of the openings has to be specially considered on the basis of the stresses due to longitudinal hull girder bending, torsion, shear and transverse loads by direct calculations.

An distribution of notch stresses can be evaluated by means of finite element calculations. For fatigue investigations the stress increase due to geometry of cut-outs has to be considered.

APPENDIX B BUCKLING

1 General

1.1 Calculation method

The calculation method for buckling strength used in the following is based on the standard DIN 18800.

1.2 Definitions

A_e = effective area for calculation of plastic section moduli [mm^2]

A_x, A_y = sectional area of longitudinal or transverse stiffeners respectively [mm^2]

a = length of single or partial plate field [mm]

b = breadth of single plate field [mm]

α = aspect ratio of single plate field

= $\frac{a}{b}$

c_s = factor accounting for the boundary conditions of the transverse stiffener

= 1.0 for simply supported stiffeners

= 2.0 for partially constraint stiffeners

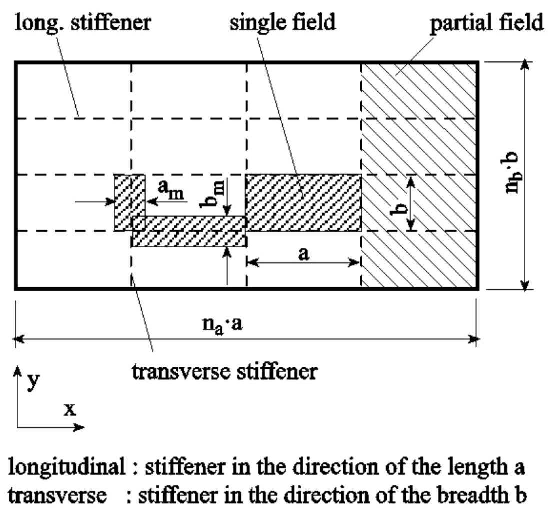


Figure 1 System of longitudinal and transverse stiffeners

f_p = shape factor (ratio of plastic and elastic section modulus of the profile), see Sec.3 [6]

h_{wx}, h_{wy} = web height [mm] of longitudinal or transverse stiffeners

I_x, I_y = moments of inertia [cm^4] of longitudinal or transverse stiffeners including effective width of plating according to [2.2]

M_0 = bending moment due to deformation w_0 of longitudinal or transverse stiffeners [Nmm]

M_1 = bending moment due to lateral load p [Nmm]

n_a, n_b = number of single plate field breadth within the partial or total plate field, see Figure 1

t = nominal plate thickness [mm]

= $t_a - t_c$

t_a = plate thickness as built [mm]

t_c = corrosion addition according to Sec.3 [5.1] [mm]

t_{wx}, t_{wy} = web thickness [mm] of longitudinal or transverse stiffeners

w_0 = assumed imperfection [mm]

w_1 = deformation of stiffener due to lateral load p at midpoint of stiffener span [mm]

ε = degree of restraint

κ = reduction factor for torsion

σ_x = membrane stress in x-direction [MPa]

σ_y = membrane stress in y-direction [MPa]

τ = shear stress in the x-y plane [MPa]

y_m = safety factor, defined as:

- 1.1 = in general
- 1.2 = for structures which are exclusively exposed to local loads
- 1.05 = for combination of statistically independent loads

Compressive and shear stresses shall be taken positive, tensile stresses shall be taken negative.

Guidance note:

If the stresses in the x- and y-direction contain already the Poisson-effect, the following modified stress values may be used:

$$\sigma_x = \frac{\sigma_x^* - 0.3 \cdot \sigma_y^*}{0.91} \qquad \sigma_y = \frac{\sigma_y^* - 0.3 \cdot \sigma_x^*}{0.91}$$

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

σ_x^*, σ_y^* = stresses containing the Poisson-effect

ψ = edge stress ratio according to Table 3

F_1 = correction factor for boundary condition at the longitudinal stiffeners according to Table 1

σ_e = reference stress

= $\frac{\pi^2}{12\left(1-\nu^2\right)} \cdot E \cdot \left(\frac{t}{b}\right)^2$ [MPa]

λ = reference degree of slenderness

= $\sqrt{\frac{R_y}{K \cdot \sigma_e}}$

K = buckling factor according to Table 3 and Table 4

In general, the ratio plate field breadth to plate thickness shall not exceed b/t = 100.

Table 1 Correction factor F_1 for boundary conditions

End form of stiffeners	*Profile type*	F_1
Sniped at both ends or single side weld	All types	1.00
Both ends are effectively connected to adjacent structures	Flat bars	1.05 [1]
	Bulb sections	1.10 [1]
	Angle and tee sections	1.20 [1]
	Girders of high rigidity, e.g. bottom transverses	1.30 [1]
1) Only guidance values, exact values may be determined by direct calculations		

2 Single plate buckling

2.1

Proof shall be provided that the following condition is complied with for the single plate field $a \cdot b$:

$$\left(\frac{\left|\sigma_x\right| \cdot \gamma_m}{\kappa_x \cdot R_y}\right)^{e1} + \left(\frac{\left|\sigma_y\right| \cdot \gamma_m}{\kappa_y \cdot R_y}\right)^{e2} - B \cdot \left(\frac{\sigma_x \cdot \sigma_x \cdot \gamma_m^2}{R_y^2}\right) + \left(\frac{\left|\tau\right| \cdot \gamma_m \cdot \sqrt{3}}{\kappa_y \cdot R_y}\right)^{e3}$$

Each term of the above condition shall be less than 1.0.

The reduction factors κ_x, κ_y and κ_τ are given in Table 3 and/or Table 4.

Where $\sigma_x \leq 0$ (tension stress), κ_x = 1.0.

Where $\sigma_y \leq 0$ (tension stress), κ_y = 1.0.

The exponents e_1, e_2 and e_3 as well as the factor B are calculated or set respectively according to Table 2.

2.2 Effective width of plating

The effective width of plating may be determined by the following equations, see also Figure 1:

$\kappa_x \cdot b$ for longitudinal stiffeners

$\kappa_y \cdot a$ for transverse stiffeners.

Table 2 Exponents e_1 – e_3 and factor B

Exponents e_1 – e_3 and factor B	*Plate field*	
	Plane	*Curved*
e_1	$1 + \kappa_x{}^4$	1.25
e_2	$1 + \kappa_y{}^4$	1.25
e_3	$1 + \kappa_x \cdot \kappa_y \cdot \kappa_\tau{}^2$	2.0

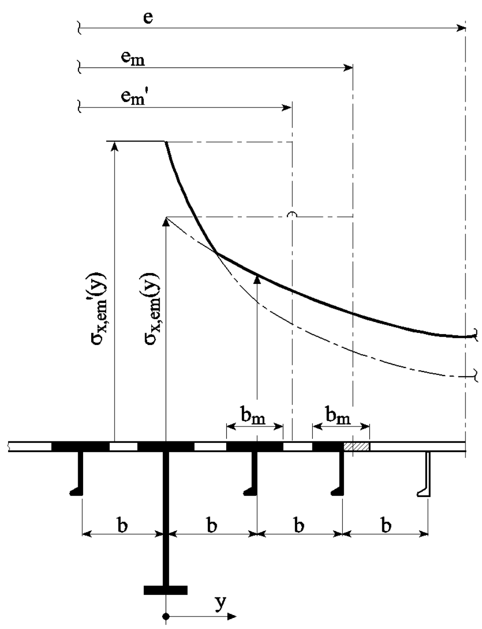
Exponents $e_1 - e_3$ and factor B	*Plate field*	
	Plane	*Curved*
B $\sigma_x\ or\ \sigma_y$ positive (compression stress)	$(\kappa_x \cdot \kappa_y)^5$	0
B $\sigma_x\ or\ \sigma_y$ negative (tension stress)	1	—

The effective width of plating shall not be taken greater than the value obtained from App.A [7.2.1].

Guidance note:

The effective width e'_m of stiffened flange plates of girders may be determined as follows:

Stiffening parallel to web of girder:



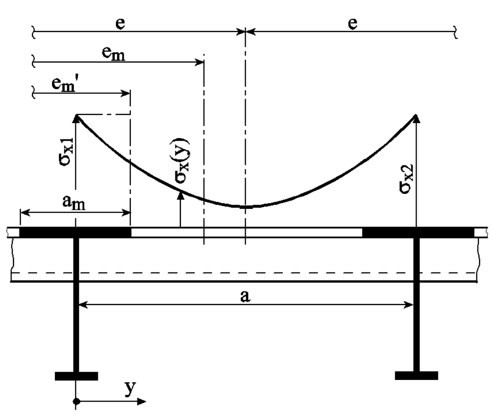
$B < e_m$

$E'_m \quad = \quad n \cdot a_m < e_m$

$n \quad = \quad$ integer number of the stiffener spacing b inside the effective breadth e_m according to App.A Table 5

$\quad = \quad \text{int}\left(\frac{e_m}{b}\right)$

Stiffening perpendicular to web of girder:



$a \geq e_m$

$e'_m \quad = \quad n \cdot a_m < e_m$

n = $2.7 \cdot \frac{e_m}{a} \leq 1$

e = width of plating supported according to App.A [7.2.1].

For $b \geq e_m$ or $a < e_m$ respectively, b and a have to be exchanged.

a_m and b_m for flange plates are in general to be determined for $\psi = 1$.

Stress distribution between two girders:

$$\sigma_x(y) = \sigma_{x1} \cdot \left\{1 - \frac{y}{e}\left[3 + c_1 - 4 \cdot c_2 - 2\frac{y}{e}\left(1 + c_1 - 2 \cdot c_2\right)\right]\right\}$$

where:

c_1 = $\frac{\sigma_{x2}}{\sigma_{x1}}$ $\quad 0 \leq c_1 \leq 1$

c_2 = $\frac{1.5}{e} \cdot \left(e''_{m1} + e''_{m2}\right) - 0.5$

e''_{m1} = $\frac{e'_{m1}}{e_{m1}}$

e''_{m2} = $\frac{e'_{m2}}{e_{m2}}$

σ_{x1}, σ_{x2} = normal stresses in flange plates of adjacent girder 1 and 2 with spacing e

y = distance of considered location from girder 1.

Scantlings of plates and stiffeners are in general to be determined according to the maximum stresses $\sigma_x(y)$ at girder webs and stiffeners respectively. For stiffeners under compression arranged parallel to the girder web with spacing b no lesser value than $0.25 \cdot R_y$ shall be inserted for $\sigma_x(y=b)$.

Shear stress distribution in the flange plates may be assumed linearly.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

2.3 Webs and flanges

For non-stiffened webs and flanges of sections and girders proof of sufficient buckling strength as for single plate fields shall be provided according to [2.1].

Guidance note:

The following guidance values are recommended for the ratio web depth to web thickness and/or flange breadth to flange thickness for normal and higher strength hull structural steel:

flat bars: $\frac{h_w}{t_w} \le \frac{215}{\sqrt{R_y}}$

angle-, tee and bulb sections:

web: $\frac{h_w}{t_w} \le \frac{661}{\sqrt{R_y}}$

flange: $\frac{b_i}{t_w} \le \frac{215}{\sqrt{R_y}}$

$b_i = b_1$ or b_2 according to Figure 2, the larger value shall be taken.

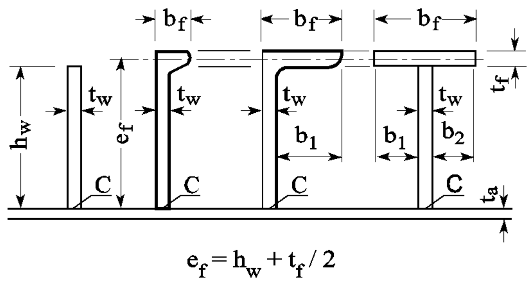


Figure 2 Main parameters of typical sections

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

3 Proof of partial and total fields

Proof shall be provided that the continuous longitudinal and transverse stiffeners of partial and total plate fields comply for lateral and torsional buckling with the conditions of Table 4.

The parameters of typical sections are shown in Figure 2 and the resulting moments of inertia are summarized in Table 5.

3.1 Longitudinal and transverse stiffeners

Proof shall be provided that the continuous longitudinal and transverse stiffeners of partial and total plate fields comply with the conditions set out in [3.2] and [3.3].

3.2 Lateral buckling

$$\frac{\sigma_a + \sigma_b}{R_y} \cdot \gamma_m \le 1.0$$

where:

σ_a = uniformly distributed compressive stress in the direction of the stiffener axis [MPa]
= σ_x for longitudinal stiffeners
= σ_y for transverse stiffeners

σ_b = bending stress in the stiffeners

= $\frac{M_0 + M_1}{W_{st} \cdot 10^3}$ [MPa]

M_o = bending moment due to deformation w of stiffener

= $F_{Ki} \frac{p_z \cdot w}{c_f - p_z}$ [N·mm]

$(c_f - p_z) > 0$

M_1 = bending moment due to the lateral load p

for continuous longitudinal stiffeners:

= $\frac{p \cdot b \cdot a^2}{24 \cdot 10^3}$ [N·mm]

for transverse stiffeners:

= $\frac{p \cdot a \cdot (n \cdot b)^2}{c_s \cdot 8 \cdot 10^3}$ [N·mm]

p = lateral load [kPa] according to Ch.3

F_{Ki} = ideal buckling force of the stiffener [N]

F_{Kix} = $\frac{\pi^2}{a^2} \cdot E \cdot I_x \cdot 10^4$ for longitudinal stiffeners

F_{Kiy} = $\frac{\pi^2}{(n \cdot b)^2} \cdot E \cdot I_y \cdot 10^4$ for transverse stiffeners

I_x, I_y = moments of inertia of the longitudinal or transverse stiffener including effective width of plating according to [2.2] [cm^4]

$$I_x \geq \frac{b \cdot t^3}{12 \cdot 10^4}$$

$$I_y \geq \frac{a \cdot t^3}{12 \cdot 10^4}$$

p_z = nominal lateral load of the stiffener due to σ_x, σ_y and τ [MPa]

for longitudinal stiffeners:

$$p_{zx} = \frac{t_a}{b}\left[\sigma_{x1}\left(\frac{\pi \cdot b}{a}\right)^2 + 2 \cdot c_y \cdot \sigma_y + \sqrt{2}\tau_1\right]$$

for transverse stiffeners:

$$p_{zy} = \frac{t_a}{a}\left[2 \cdot c_x \cdot \sigma_{x1} + \sigma_y\left(\frac{\pi \cdot a}{n \cdot b}\right)^2\left(1 + \frac{A_y}{a \cdot t_a}\right) + \sqrt{2}\tau_1\right]$$

$$\sigma_{x1} = \sigma_x\left(1 + \frac{A_x}{b \cdot t_a}\right)$$

c_x, c_y = factor taking into account the stresses vertical to the stiffener's axis and distributed variable along the stiffener's length

= $0.5\,(1 + \psi)$ for $0 \leq \psi \leq 1$

= $\frac{0.5}{1 - \psi}$ for $\psi < 0$

ψ = edge stress ratio according to Table 3

A_x, A_y = sectional area of the longitudinal or transverse stiffener respectively [mm^2]

τ_1 = $\left[\tau - t \cdot \sqrt{R_y \cdot E \cdot \left(\frac{m_1}{a^2} + \frac{m_2}{b^2}\right)}\right] \geq 0$

for longitudinal stiffeners:

$\frac{a}{b} \geq 2.0 : m_1 = 1.47 \qquad m_2 = 0.49$

$\frac{a}{b} < 2.0 : m_1 = 1.96 \qquad m_2 = 0.37$

for transverse stiffeners:

$\frac{a}{n \cdot b} \geq 0.5 : m_1 = 0.37 \qquad m_2 = \frac{1.96}{n^2}$

$\frac{a}{n \cdot b} < 0.5 : m_1 = 0.49 \qquad m_2 = \frac{1.47}{n^2}$

w = $w_o + w_1$

w_o = assumed imperfection [mm]

$\frac{a}{250} \geq w_{ox} \leq \frac{b}{250}$ for longitudinal stiffeners

$\frac{n \cdot b}{250} \geq w_{oy} \leq \frac{a}{250}$ for transverse stiffeners

however $w_o \leq 10$ mm

Guidance note:

For stiffeners sniped at both ends w_o shall not be taken less than the distance from the midpoint of plating to the neutral axis of the profile including effective width of plating.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

w_1 = deformation of stiffener due to lateral load p at midpoint of stiffener span [mm].

In case of uniformly distributed load the following values for w_1 may be used:

for longitudinal stiffeners:

$$w_1 = \frac{p \cdot b \cdot a^4}{384 \cdot 10^7 \cdot E \cdot I_x}$$

for transverse stiffeners:

$$w_1 = \frac{5 \cdot a \cdot p(n \cdot b)^4}{384 \cdot 10^7 \cdot E \cdot I_y \cdot c_s^2}$$

c_f = elastic support provided by the stiffener [MPa]

c_{fx} = $F_{Kix} \cdot \frac{\pi^2}{a^2} \cdot \left(1 + c_{px}\right)$ for longitudinal stiffeners

c_{px} = $\frac{1}{1 + \frac{0.91}{c_{x\alpha}} \cdot \left(\frac{12 \cdot 10^4 \cdot I_x}{t^3 \cdot b} - 1\right)}$

$c_{x\alpha}$ = $\left[\frac{a}{2b}+\frac{2b}{a}\right]^2$ for $a \geq 2b$

= $\left[1+\left(\frac{a}{2b}\right)^2\right]^2$ for $a < 2b$

c_{fy} = $c_s \cdot F_{Kiy} \cdot \frac{\pi^2}{(n \cdot b)^2} \cdot \left(1+c_{py}\right)$

for transverse stiffeners

c_s = factor accounting for the boundary conditions of the transverse stiffener

= 1.0 for simply supported stiffeners

= 2.0 for partially constraint stiffeners

c_{py} = $\frac{1}{1+\frac{0.91}{c_{y\alpha}} \cdot \left(\frac{12 \cdot 10^4 \cdot I_y}{t^3 \cdot a}-1\right)}$

$c_{y\alpha}$ = $\left[\frac{n \cdot b}{2a}+\frac{2a}{n \cdot b}\right]^2$ for $n \cdot b \geq 2a$

= $\left[1+\left(\frac{n \cdot b}{2a}\right)^2\right]^2$ for $n \cdot b < 2a$

W_{st} = section modulus of stiffener (longitudinal or transverse) [cm^3] including effective width of plating according to [2.2].

If no lateral load *p* is acting the bending stress σ_b shall be calculated at the midpoint of the stiffener span for that fibre which results in the largest stress value. If a lateral load p is acting, the stress calculation shall be carried out for both fibres of the stiffener's cross sectional area (if necessary for the biaxial stress field at the plating side).

Guidance note:

Longitudinal and transverse stiffeners not subjected to lateral load *p* have sufficient scantlings if their moments of inertia I_x and I_y are not less than obtained by the following equations:

$$I_x = \frac{p_{zx} a^2}{\pi^2 \cdot 10^4} \cdot \left(\frac{w_{0x} \cdot h_w}{\frac{R_y}{\gamma_m}-\sigma_x}+\frac{a^2}{\pi^2 \cdot E}\right) \text{ [cm}^4\text{]}$$

$$I_y = \frac{p_{zy} (n \cdot b)^2}{\pi^2 \cdot 10^4} \cdot \left(\frac{w_{0y} \cdot h_w}{\frac{R_y}{\gamma_m}-\sigma_y}+\frac{(n \cdot b)^2}{\pi^2 \cdot E}\right) \text{ [cm}^4\text{]}$$

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

Table 3 Plane plate fields

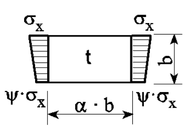
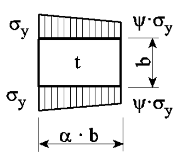
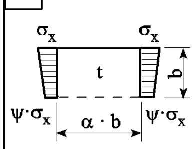
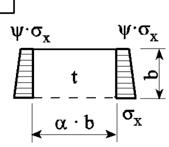
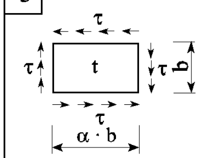
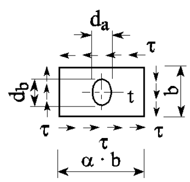
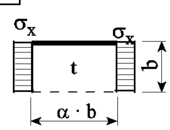
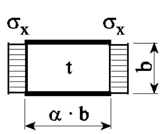
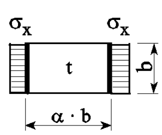
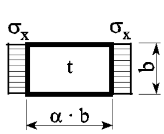
Load case	Edge stress ratio ψ	Aspect ratio α	Buckling factor K	Reduction factor κ
1 	$1 \ge \psi \ge 0$	$\alpha > 1$	$K = \frac{8.4}{\psi + 1.1}$	$\kappa_x = 1$ for $\lambda \le \lambda_c$
	$0 > \psi > -1$		$K = 7.63 - \psi\,(6.26 - 10\,\psi)$	$\kappa_x = c\left(\frac{1}{\lambda} - \frac{0.22}{\lambda^2}\right)$ for $\lambda > \lambda_c$ $c = 1.25 - 0.12\psi \le 1.25$
	$\psi \le -1$		$K = (1-\psi)^2 \cdot 5.975$	$\lambda_c = \frac{c}{2}\left(1 + \sqrt{1 - \frac{0.88}{c}}\right)$
2 	$1 \ge \psi \ge 0$	$\alpha \ge 1$	$K = F_1\left(1 + \frac{1}{\alpha^2}\right)^2 \frac{2.1}{(\psi+1.1)}$	$\kappa_y = c\left(\frac{1}{\lambda} - \frac{R+F^2\,(H-R)}{\lambda^2}\right)$ $c = 1.25 - 0.12\psi \le 1.25$
	$0 > \psi > -1$	$1 \le \alpha \le 1.5$	$K = F_1\left[\left(1 + \frac{1}{\alpha^2}\right)^2 \frac{2.1\,(1+\psi)}{1.1} - \frac{\psi}{\alpha^2}\,(13.9 - 10\,\psi)\right]$	$R = \lambda\left(1 - \frac{\lambda}{c}\right)$ for $\lambda < \lambda_c$ $R = 0.22$ for $\lambda \ge \lambda_c$ $\lambda_c = \frac{c}{2}\left(1 + \sqrt{1 - \frac{0.88}{c}}\right)$
		$\alpha > 1.5$	$K = F_1\left[\left(1 + \frac{1}{\alpha^2}\right)^2 \frac{2.1\,(1+\psi)}{1.1} - \frac{\psi}{\alpha^2}\,(5{,}87 + 1{,}87\,\alpha^2 + \frac{8.6}{\alpha^2} - 10\,\psi)\right]$	$F = \left(1 - \frac{\frac{K}{0.91} - 1}{\lambda_p^2}\right) c_1 \ge 0$ $\lambda_p^2 = \lambda^2 - 0.5$ $\quad 1 \le \lambda_p^2 \le 3$ $c_1 = 1$ for σ_y due to direct loads
	$\psi \le -1$	$1 \le \alpha \le \frac{3\,(1-\psi)}{4}$	$K = F_1\left(\frac{1-\psi}{\alpha}\right)^2 5.975$	$c_1 = \left(1 - \frac{F_1}{\alpha}\right) \ge 0$ for σ_y due to bending (in general)
		$\alpha > \frac{3\,(1-\psi)}{4}$	$K = F_1\left[\left(\frac{1-\psi}{\alpha}\right)^2 3.9675 + 0.5375\left(\frac{1-\psi}{\alpha}\right)^4 + 1{,}87\right]$	$c_1 = 0$ for σ_y due to bending in extreme load cases (e. g. w. t. bulkheads) $H = \lambda - \frac{2\lambda}{c\,(T + \sqrt{T^2 - 4})} \ge R$ $T = \lambda + \frac{14}{15\lambda} + \frac{1}{3}$
3 	$1 \ge \psi \ge 0$	$\alpha > 0$	$K = \frac{4\,(0.425 + 1/\alpha^2)}{3\,\psi + 1}$	
	$0 > \psi \ge -1$		$K = 4\left(0.425 + \frac{1}{\alpha^2}\right)(1+\psi) - 5 \cdot \psi\,(1 - 3.42\,\psi)$	$k_x = 1$ for $l \le 0.7$
4 	$1 \ge \psi \ge -1$	$\alpha > 0$	$K = \left(0.425 + \frac{1}{\alpha^2}\right)\frac{3 - \psi}{2}$	$k_x = \frac{1}{\lambda^2 + 0.51}$ for $l > 0.7$

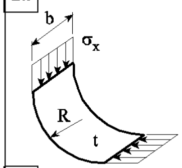
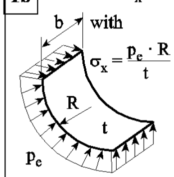
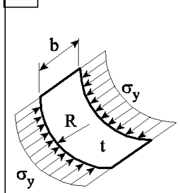
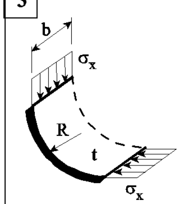
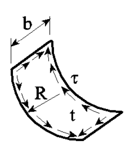
Table 3 Plane plate fields (continued)

Load case	Edge stress ratio ψ	Aspect ratio α	Buckling factor K	Reduction factor κ
5 	—		$K = K_\tau \cdot \sqrt{3}$	
		$\alpha \geq 1$	$K_\tau = 5.34 + \frac{4}{\alpha^2}$	
		$0 < \alpha < 1$	$K_\tau = 4 + \frac{5.34}{\alpha^2}$	$\kappa_\tau = 1$ for $\lambda \leq 0.84$ $\kappa_\tau = \frac{0.84}{\lambda}$ for $\lambda > 0.84$
6 	—		$K = K' \cdot r$ K' = K according to load case 5 r = Reduction factor $r = \left(1 - \frac{d_a}{a}\right)\left(1 - \frac{d_b}{b}\right)$ with $\frac{d_a}{a} \leq 0.7$ and $\frac{d_b}{b} \leq 0.7$	
7 	—	$\alpha \geq 1.64$	$K = 1.28$	$\kappa_x = 1$ for $\lambda \leq 0.7$
		$\alpha < 1.64$	$K = \frac{1}{\alpha^2} + 0.56 + 0.13\,\alpha^2$	$\kappa_x = \frac{1}{\lambda^2 + 0.51}$ for $\lambda > 0.7$
8 	—	$\alpha \geq \frac{2}{3}$	$K = 6.97$	
		$\alpha < \frac{2}{3}$	$K = \frac{1}{\alpha^2} + 2.5 + 5\,\alpha^2$	
9 	—	$\alpha \geq 4$	$K = 4$	$\kappa_x = 1$ for $\lambda \leq 0.83$
		$4 > \alpha > 1$	$K = 4 + \left[\frac{4-\alpha}{3}\right]^4 2.74$	$\kappa_x = 1.13\left[\frac{1}{\lambda} - \frac{0.22}{\lambda^2}\right]$ for $\lambda > 0.83$
		$\alpha \leq 1$	$K = \frac{4}{\alpha^2} + 2.07 + 0.67\,\alpha^2$	
10 	—	$\alpha \geq 4$	$K = 6.97$	
		$4 > \alpha > 1$	$K = 6.97 + \left[\frac{4-\alpha}{3}\right]^4 3.1$	
		$\alpha \leq 1$	$K = \frac{4}{\alpha^2} + 2.07 + 4\,\alpha^2$	

Explanations for boundry conditions

- - - - - - plate edge free
——— plate edge simply supported
▬▬▬ plate edge clamped

Table 4 Curved plate field $R/t \le 2500$[1]

Load case	Aspect ratio b / R	Buckling factor K	Reduction factor κ
1a 	$\frac{b}{R} \le 1.63\sqrt{\frac{R}{t}}$	$K = \frac{b}{\sqrt{R \cdot t}} + 3\,\frac{(R \cdot t)^{0.175}}{b^{0.35}}$	$\kappa_x = 1$, for $\lambda \le 0.4$ [2] $\kappa_x = 1.274 - 0.686\,\lambda$ for $0.4 < \lambda \le 1.2$ $\kappa_x = \frac{0.65}{\lambda^2}$, for $\lambda > 1.2$
1b  p_e = external pressure in [N/mm²]	$\frac{b}{R} > 1.63\sqrt{\frac{R}{t}}$	$K = 0.3\,\frac{b^2}{R^2} + 2.25\left(\frac{R^2}{b \cdot t}\right)^2$	
2 	$\frac{b}{R} \le 0.5\sqrt{\frac{R}{t}}$	$K = 1 + \frac{2}{3}\,\frac{b^2}{R \cdot t}$	$\kappa_y = 1$, for $\lambda \le 0.25$ [2] $\kappa_y = 1.233 - 0.933\,\lambda$ for $0.25 < \lambda \le 1$ $\kappa_y = 0.3 / \lambda^3$ for $1 < \lambda \le 1.5$ $\kappa_y = 0.2 / \lambda^2$ for $\lambda > 1.5$
	$\frac{b}{R} > 0.5\sqrt{\frac{R}{t}}$	$K = 0.267\,\frac{b^2}{R \cdot t}\left[3 - \frac{b}{R}\sqrt{\frac{t}{R}}\right] \ge 0.4\,\frac{b^2}{R \cdot t}$	
3 	$\frac{b}{R} \le \sqrt{\frac{R}{t}}$	$K = \frac{0.6 \cdot b}{\sqrt{R \cdot t}} + \frac{\sqrt{R \cdot t}}{b} - 0.3\,\frac{R \cdot t}{b^2}$	as in load case 1a
	$\frac{b}{R} > \sqrt{\frac{R}{t}}$	$K = 0.3\,\frac{b^2}{R^2} + 0.291\left(\frac{R^2}{b \cdot t}\right)^2$	
4 	$\frac{b}{R} \le 8.7\sqrt{\frac{R}{t}}$	$K = K_\tau \cdot \sqrt{3}$ $K_\tau = \left[28.3 + \frac{0.67 \cdot b^3}{R^{1.5} \cdot t^{1.5}}\right]^{0.5}$	$\kappa_\tau = 1$, for $\lambda \le 0.4$ $\kappa_\tau = 1.274 - 0.686\,\lambda$ for $0.4 < \lambda \le 1.2$ $\kappa_\tau = \frac{0.65}{\lambda^2}$ for $\lambda > 1.2$
	$\frac{b}{R} > 8.7\sqrt{\frac{R}{t}}$	$K_\tau = 0.28\,\frac{b^2}{R\sqrt{R \cdot t}}$	

Explanations for boundry conditions:
- - - - - - plate edge free
——— plate edge simply supported
━━━ plate edge clamped

1 For curved plate fields with a very large radius the κ-value need not to be taken less than one derived for the expanded plane field.

2 For curved single fields, e.g. the bilge strake, which are located within plane partial or total fields, the reduction factor κ may taken as follows:

Load case 1b: $\kappa_x = 0.8/\lambda^2 \le 1.0$; load case 2: $\kappa_y = 0.65/\lambda^2 \le 1.0$

3.3 Torsional buckling

3.3.1 Longitudinal stiffeners

$$\frac{\sigma_x \cdot \gamma_m}{\kappa_T \cdot R_y} \leq 1.0$$

κ_T = 1.0 for $\lambda_T \leq 0.2$

= $\dfrac{1}{\phi + \sqrt{\phi^2 - \lambda_T{}^2}}$ for $\lambda_T > 0.2$

$$0.5\left(1 + 0.21\left(\lambda_T - 0.2\right) + \lambda_T{}^2\right)$$

λ_T = reference degree of slenderness

= $\sqrt{\dfrac{R_y}{\sigma_{KiT}}}$

$$\frac{E}{I_p}\left(\frac{\pi^2 \cdot I_\omega \cdot 10^2}{a^2}\varepsilon + 0.385 \cdot I_T\right)[\mathrm{MPa}]$$

For I_P, I_T, I_ω see Table 5.

I_P = polar moment of inertia of the stiffener related to the point C [cm^4]
I_T = St. Venant's moment of inertia of the stiffener [cm^4]
I_ω = sectorial moment of inertia of the stiffener related to the point C [cm^6]
ε = degree of fixation

= $1 + 10^{-4}\sqrt{\dfrac{a^4}{I_\omega\left(\dfrac{b}{t^3} + \dfrac{4h_w}{3t_w^3}\right)}}$

h_w = web height [mm]
t_w = web thickness [mm]
b_f = flange breadth [mm]
t_f = flange thickness [mm].

3.3.2 Transverse stiffeners

For transverse stiffeners loaded by compressive stresses and which are not supported by longitudinal stiffeners, proof shall be provided in accordance with [3.3.1] analogously.

Table 5 Geometric properties of typical sections

Section	I_P	I_T	I_ω
Flat bar	$\frac{h_w^3 \cdot t_w}{3 \cdot 10^4}$	$\frac{h_w \cdot t_w^3}{3 \cdot 10^4}\left(1 - 0.63\frac{t_w}{h_w}\right)$	$\frac{h_w^3 \cdot t_w^3}{36 \cdot 10^6}$
Sections with bulb or flange	$\left(\frac{A_w \cdot h_w^2}{3} + A_f \cdot e_f^2\right)10^{-4}$	$\frac{h_w \cdot t_w^3}{3 \cdot 10^4}\left(1 - 0.63\frac{t_w}{h_w}\right)$ + $\frac{b_f \cdot t_f^2}{3 \cdot 10^4}\left(1 - 0.63\frac{t_f}{b_f}\right)$	for bulb and angle sections: $\frac{A_f \cdot c_f^2 \cdot b_f^2}{12 \cdot 10^6}\left(\frac{A_f + 2.6\ A_w}{A_f + A_w}\right)$ for tee-sections: $\frac{b_f^3 \cdot t_f \cdot e_f^2}{12 \cdot 10^6}$
Web area: $A_w = h_w \cdot t_w$ Flange area: $A_f = b_f \cdot t_f$			

APPENDIX C WELDED JOINTS

1 General

1.1 Information contained in manufacturing documents

1.1.1 The shapes and dimensions of welds and, where proof by calculation is supplied, the requirements applicable to welded joints (the weld quality grade, detail category) shall be stated in drawings and other manufacturing documents (parts lists, welding and inspection schedules). In special cases, e.g. where special materials are concerned, the documents shall also state the welding method, the welding consumables used, heat input and control, the weld build-up and any post-weld treatment which may be required.

1.1.2 Symbols and signs used to identify welded joints shall be explained if they depart from the symbols and definitions contained in the relevant standards (e.g. DIN standards). Where the weld preparation (together with approved methods of welding) conforms both to normal shipbuilding practice and to these rules and recognized standards, where applicable, no special description is needed.

1.2 Materials, weldability

1.2.1 Only base materials of proven weldability, see Pt.2, may be used for welded structures. Any approval conditions of the steel or of the procedure qualification tests and the steel maker's recommendations shall be observed.

1.2.2 For normal strength hull structural steels grades A, B, D and E which have been tested by the Society, weldability is considered to have been proven. The suitability of these base materials for high efficiency welding processes with high heat input shall be verified.

1.2.3 Higher strength hull structural steels grade AH/DH/EH/FH which have been approved by the Society in accordance with the relevant requirements of the rules for materials and welding normally have had their weldability examined and, provided their handling is in accordance with normal shipbuilding practice, may be considered to be proven. The suitability of these base materials for high efficiency welding processes with high heat input shall be verified.

1.2.4 High strength (quenched and tempered) fine grain structural steels, low temperature steels, stainless and other (alloyed) structural steels require special approval by the Society. Proof of weldability of the respective steel shall be presented in connection with the welding procedure and the welding consumables.

1.2.5 Cast steel and forged parts shall comply with the rules for materials and shall have been tested by the Society. The carbon content of components made from carbon and carbon-manganese steels/castings for welded structures shall not exceed 0.23% C at ladle analysis (piece analysis max. 0.25% C).

1.2.6 Light metal alloys shall have been tested by the Society in accordance with the rules for materials. Their weldability shall have been verified in combination with welding processes and welding consumables. It can generally be taken for granted in the case of the alloys mentioned in the rules for materials.

1.2.7 Welding consumables used shall be suitable for the parent metal to be welded and shall be approved by the Society. Where filler materials having strength properties deviating (downwards) from the parent metal are used (upon special agreement by the Society), this has to be taken into account when dimensioning the welded joints.

1.3 Manufacture and testing

1.3.1 The manufacture of welded structural components may only be carried out in workshops or plants that have been approved. The requirements that have to be observed in connection with the fabrication of welded joints are laid down in the Pt.2 apply.

1.3.2 The weld quality grade of welded joints without proof by calculation, see [1.1.1], depends on the significance of the welded joint for the total structure and on its location in the structural element (location to the main stress direction) and on its stressing. For details concerning the type, scope and manner of testing, see the rules for Pt.2 Where proof of fatigue strength is required in addition, the requirements of App.D apply.

2 Design

2.1 General design principles

2.1.1 During the design stage welded joints shall be planned such as to be accessible during fabrication, to be located in the best possible position for welding and to permit proper welding sequence to be followed.

2.1.2 Both the welded joints and the sequence of welding involved shall be so planned as to enable residual welding stresses to be kept to a minimum in order that no excessive deformation occurs. Welded joints should not be over dimensioned, see also [2.3.3].

2.1.3 When planning welded joints, it shall first be established that the type and grade of weld envisaged, such as full root weld penetration in the case of HV or DHV (K) weld seams, can in fact be perfectly executed under the conditions set by the limitations of the manufacturing process involved. If this is not the case, a simpler type of weld seam shall be selected and its possibly lower load bearing capacity taken into account when dimensioning the component.

2.1.4 Highly stressed welded joints - which, therefore, are generally subject to examination - shall be so designed that the most suitable method of testing for faults can be used (radiography, ultrasonic, surface crack testing methods) in order that a reliable examination may be carried out.

2.1.5 Special characteristics peculiar to the material, such as the lower strength values of rolled material in the thickness direction (see [2.2.5]) or the softening of cold worked aluminium alloys as a result of welding, are factors which have to be taken into account when designing welded joints. Clad plates where the efficiency of the bond between the base and the clad material is proved may generally be treated as solid plates (up to medium plate thicknesses where mainly fillet weld connections are used).

2.1.6 In cases where different types of material are paired and operate in sea water or any other electrolytic medium, for example welded joints made between unalloyed carbon steels and stainless steels in the wear-resistant cladding in rudder nozzles or in the cladding of rudder shafts, the resulting differences in potential greatly increase the susceptibility to corrosion and shall, therefore, be given special attention. Where possible, such welds shall be positioned in locations less subject to the risk of corrosion (such as on the outside of tanks) or special protective counter-measures shall be taken (such as the provision of a protective coating or cathodic protection).

2.2 Design details

2.2.1 Stress flow, transitions

All welded joints on primary supporting members shall be designed to provide as smooth a stress profile as possible with no major internal or external notches, no discontinuities in rigidity and no obstructions to strains, see App.A [8.5.3].

This applies in analogous manner to the welding of subordinate components on to primary supporting members whose exposed plate or flange edges should, as far as possible, be kept free from notch effects due to welded attachments. Regarding the inadmissibility of weldments to the upper edge of the sheer strake, see Ch.4 Sec.3 [8]. This applies similarly to weldments to the upper edge of continuous side coamings of large openings.

Butt joints in long or extensive continuous structures such as bilge keels, fenders, crane rails, slop coamings, etc. attached to primary structural members are therefore to be welded over their entire cross-section.

Wherever possible, joints (especially site joints) in girders and sections shall not be located in areas of high bending stress. Joints at the knuckle of flanges shall be avoided.

The transition between differing component dimensions shall be smooth and gradual. Where the depths of web of girders or sections differ, the flanges or bulbs shall be bevelled and the web slit and expanded or pressed together to equalize the depths of the members. The length of the transition should be at least equal twice the difference in depth.

Where the plate thickness differs at joints perpendicularly to the direction of the main stress, differences in thickness greater than 3 mm shall be accommodated by bevelling the proud edge in the manner shown in Figure 1 at an effective ratio of at least 1 : 3 or according to the notch category. Differences in thickness of 3 mm but not more than half of the thinner plate thickness may be accommodated within the weld.

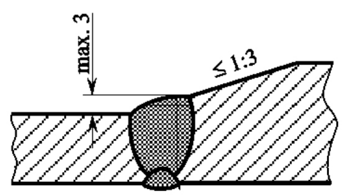


Figure 1 Accommodation of differences of thickness

For the welding on of plates or other relatively thin-walled elements, steel castings and forgings should be appropriately tapered or provided with integrally cast or forged welding flanges in accordance with Figure 2.

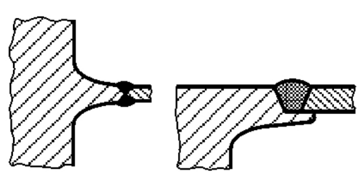


Figure 2 Welding flanges on steel castings or forgings

For the connection of shaft brackets to the boss and shell plating, see [2.4.3]; for the connection of horizontal coupling flanges to the rudder body, see [2.4.4]. For the required thickened rudderstock collar required with build-up welds and for the connection of the coupling flange, see [2.2.7] and Ch.7 Sec.2 [6.2]. Rudderstock and coupling flange shall be connected by full penetration weld, see [2.4.4].

2.2.2 Local clustering of welds, minimum spacing

The local clustering of welds and short distances between welds shall be avoided. Adjacent butt welds should be separated from each other by a distance of at least:

50 mm + 4 × plate thickness

Fillet welds should be separated from each other and from butt welds by a distance of at least:

30 mm + 2 × plate thickness

The width of replaced or inserted plates (strips) should, however, be at least 300 mm or ten times the plate thickness, whichever is the greater.

Reinforcing plates, welding flanges, mountings and similar components socket-welded into plating should be of the following minimum size:

$$D_{\text{min}} = 170 + 3(t - 10) \geq 170\text{mm}$$

where:

D	=	diameter of round or length of side of angular weldments [mm]
t	=	plating thickness [mm].

The corner radii of angular socket weldments should be 5 t [mm] but at least 50 mm. Alternatively the longitudinal seams shall extend beyond the transverse seams. Socket weldments shall be fully welded to the surrounding plating.

Regarding the increase of stress due to different thickness of plates see also App.D [2.1.5].

2.2.3 Welding cut-outs

Welding cut-outs for the (later) execution of butt or fillet welds following the positioning of transverse members should be rounded (minimum radius 25 mm or twice the plate thickness, whichever is the greater) and should be shaped to provide a smooth transition on the adjoining surface as shown in Figure 3 (especially necessary where the loading is mainly dynamic).

Where the welds are completed prior to the positioning of the crossing members, no welding cut-outs are needed. Any weld reinforcements present shall be machined off prior to the location of the crossing members or these members shall have suitable cut-outs.

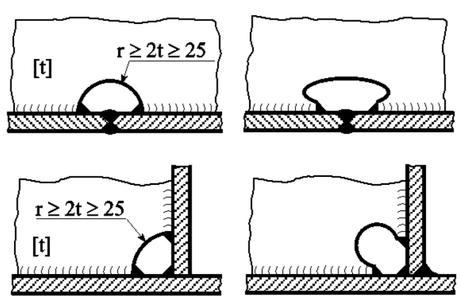


Figure 3 Welding cut-outs

2.2.4 Local reinforcements, doubling plates

Where platings (including girder plates and tube walls) are subjected locally to increased stresses, thicker plates should be used wherever possible in preference to doubling plates. Bearing bushes, hubs etc. shall invariably take the form of thicker sections welded into the plating, see [2.2.2].

Where doublings cannot be avoided, the thickness of the doubling plates should not exceed twice the plating thickness. Doubling plates whose width is greater than approximately 30 times their thickness shall be plug welded to the underlying plating in accordance with [2.3.3] at intervals not exceeding 30 times the thickness of the doubling plate.

Along their (longitudinal) edges, doubling plates shall be continuously fillet welded with a throat thickness *a* of 0.3 × the doubling plate thickness. At the ends of doubling plates, the throat thickness *a* at the end faces shall be increased to 0.5 × the doubling plate thickness but shall not exceed the plating thickness (see Figure 4).

The welded transition at the end faces of the doubling plates to the plating should form with the latter an angle of 45° or less.

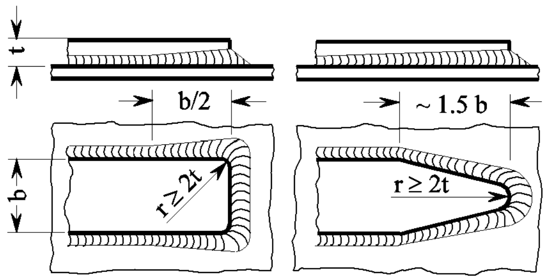


Figure 4 Welding at the ends of doubling plates

Where proof of fatigue strength is required, see App.D, the configuration of the end of the doubling plate shall conform to the selected detail category.

Doubling plates are not permitted in tanks for flammable liquids except collar plates and small doublings for fittings like tank heating fittings or fittings for ladders.

2.2.5 Intersecting members, stress in the thickness direction

Where, in the case of intersecting members, plates or other rolled products are stressed in the thickness direction by shrinking stresses due to the welding and/or applied loads, suitable measures shall be taken in the design and fabrication of the structures to prevent lamellar tearing (stratified fractures) due to the anisotropy of the rolled products.

Such measures include the use of suitable weld shapes with a minimum weld volume and a welding sequence designed to reduce transverse shrinkage. Other measures are the distribution of the stresses over a larger area of the plate surface by using a build-up weld or the joining together of several fibres members stressed in the thickness direction as exemplified by the deck stringer/sheer strake joint shown in Figure 12.

In case of very severe stresses in the thickness direction due, for example, to the aggregate effect of the shrinkage stresses of bulky single or double-bevel butt welds plus high applied loads, plates with guaranteed through thickness properties (extra high-purity material and guaranteed minimum reductions in area of tensile test specimens taken in thickness direction)[1] shall be used.

2.2.6 Welding of cold formed sections, bending radii

Wherever possible, welding should be avoided at the cold formed sections with more than 5% permanent elongation and in the adjacent areas of structural steels with a tendency towards strain ageing.

[1] See Pt.2 and also Supply Conditions 096 for Iron and Steel Products, *Plate, strip and universal steel with improved resistance to stress perpendicular to the product surface* issued by the German Iron and Steelmakers' Association.

Guidance note:

Elongation e in the outer tensile-stressed zone

$$\varepsilon = \frac{100}{1 + 2 \cdot \frac{r}{t}} [\%]$$

r = inner bending radius [mm]

t = plate thickness [mm]

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

Welding may be performed at the cold formed sections and adjacent areas of hull structural steels and comparable structural steels (e.g. those in quality groups S... J... and S... K... to DIN-EN 10025) provided that the minimum bending radii are not less than those specified in Table 1.

Note:

The bending capacity of the material may necessitate a larger bending radius.

---e-n-d---o-f---n-o-t-e---

Table 1 Minimum inner bending radii for cold formed sections

Plate thickness t	*Minimum inner bending radius r*
to 4 mm	1.0 × *t*
to 8 mm	1.5 × *t*
to 12 mm	2.0 × *t*
to 24 mm	3.0 × *t*
over 24 mm	5.0 × *t*

For other steels and other materials, where applicable, the necessary minimum bending radius shall, in case of doubt, be established by test. Proof of adequate toughness after welding may be stipulated for steels with minimum yield stress of more than 355 MPa and plate thicknesses of 30 mm and above which have undergone cold forming resulting in 2% or more permanent elongation.

2.2.7 Build-up welds on rudder stocks and pintles

Wear resistance and/or corrosion resistant build-up welds on the bearing surfaces of rudder-stocks, pintles etc. shall be applied to a thickened collar exceeding by at least 20 mm the diameter of the adjoining part of the shaft.

Where a thickened collar is impossible for design reasons, the build-up weld may be applied to the smooth shaft provided that relief-turning in accordance with the last paragraph in this section is possible (leaving an adequate residual diameter).

After welding, the transition areas between the welded and non-welded portions of the shaft shall be relief-turned with large radii, as shown in Figure 5, to remove any base material whose structure close to the concave groove has been altered by the welding operation and in order to effect the physical separation of geometrical and metallurgical notches.

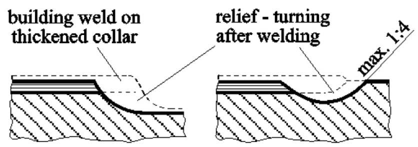


Figure 5 Build-up welds applied to rudderstocks and pintles

2.3 Weld shapes and dimensions

2.3.1 Butt joints

Depending on the plate thickness, the welding method and the welding position, butt joints shall be of the square, V or double-V shape conforming to the relevant standards (e.g. EN 22553/ISO 2533, ISO 9692 -1, -2, -3 or -4). Where other weld shapes are applied, these shall be specially described in the drawings. Weld shapes for special welding processes such as single-side or electrogas welding shall have been tested and approved in the context of a welding procedure test.

As a matter of principle, the rear sides of butt joints shall be grooved and welded with at least one capping pass. Exceptions to this rule, as in the case of submerged-arc welding or the welding processes mentioned in the first paragraph, require to be tested and approved in connection with a welding procedure test. The effective weld thickness shall be deemed to be the plate thickness, or, where the plate thicknesses differ, the lesser plate thickness. Where proof of fatigue strength is required, see App.D, the detail category depends on the execution (quality) of the weld.

Where the aforementioned conditions cannot be met, e.g. where the welds are accessible from one side only, the joints shall be executed as lesser bevelled welds with an open root and an attached or an integrally machined or cast, permanent weld pool support (backing) as shown in Figure 6.

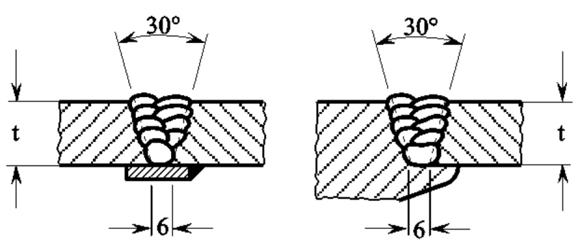


Figure 6 Single-side welds with permanent weld pool supports (backings)

The weld shapes illustrated in Figure 7 shall be used for clad plates. These weld shapes shall be used in analogous manner for joining clad plates to (unalloyed and low alloyed) hull structural steels.

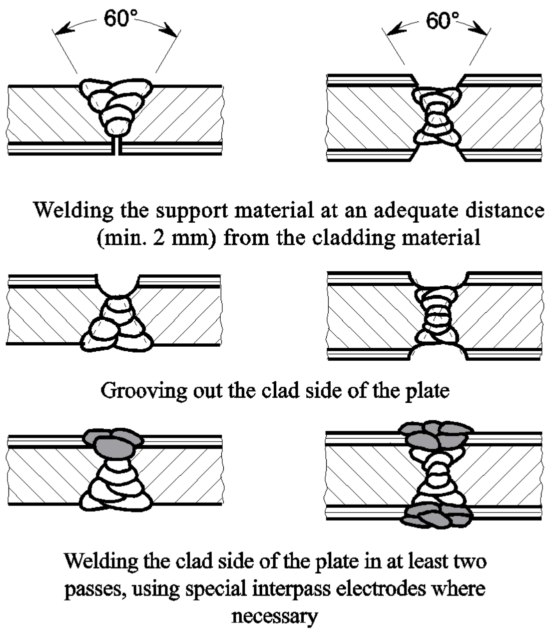


Figure 7 Weld shapes for welding of clad plates

2.3.2 Corner, T and double-T (cruciform) joints

Corner, T and double-T (cruciform) joints with complete union of the abutting plates shall be made as single or double-bevel welds with a minimum root face and adequate air gap, as shown in Figure 8, and with grooving of the root and capping from the opposite side.

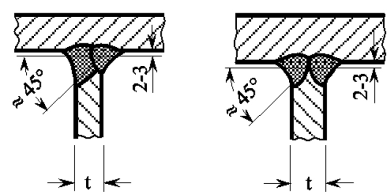


Figure 8 Single and double-bevel welds with full root penetration

The effective weld thickness shall be assumed as the thickness of the abutting plate. Where proof of fatigue strength is required, see App.D, the detail category depends on the execution (quality) of the weld.

Corner, T and double-T (cruciform) joints with a defined incomplete root penetration, as shown in Figure 9, shall be made as single or double-bevel welds, as described in the first paragraph of this section, with a back-up weld but without grooving of the root.

The effective weld thickness may be assumed as the thickness of the abutting plate t, where f is the incomplete root penetration of 0.2 t with a maximum of 3 mm, which shall be balanced by equally sized double fillet welds on each side. Where proof of fatigue strength is required, see App.D, these welds shall be assigned to type D1.

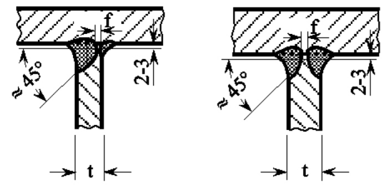


Figure 9 Single and double-bevel welds with defined incomplete root penetration

Corner, T and double-T (cruciform) joints with both an unwelded root face *c* and a defined incomplete root penetration f shall be made in accordance with Figure 10.

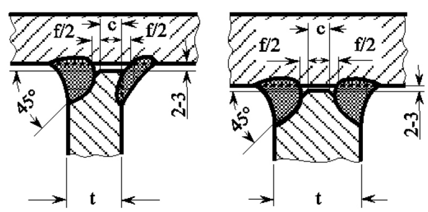


Figure 10 Single and double-bevel welds with unwelded root face and defined incomplete root penetration

The effective weld thickness shall be assumed as the thickness of the abutting plate *t* minus (*c* + *f*), where *f* shall be assigned a value of 0.2 *t* subject to a maximum of 3 mm. Where proof of fatigue strength is required, see App.D, these welds shall be assigned to types D2 or D3.

Corner, T and double-T (cruciform) joints which are accessible from one side only may be made in accordance with Figure 11 in a manner analogous to the butt joints referred to in [2.3.1] using a weld pool support (backing), or as single-side, single bevel welds in a manner similar to those prescribed previously.

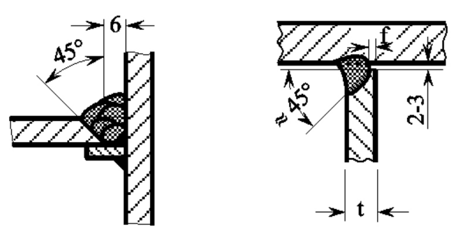


Figure 11 Single-side welded T joints

The effective weld thickness shall be determined by analogy with [2.3.1] or [2.3.2], as appropriate. Wherever possible, these joints should not be used where proof of fatigue strength is required, see App.D.

Where corner joints are flush, the weld shapes shall be as shown in Figure 12 with bevelling of at least 30° of the vertically drawn plates to avoid the danger of lamellar tearing. A similar procedure shall be followed in the case of fitted T joints (uniting three plates) where the abutting plate shall be socketed between the aligned plates.

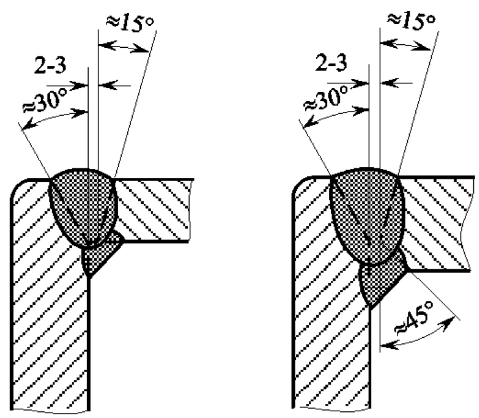


Figure 12 Flush fitted corner joints

Where, in the case of T joints, the direction of the main stress lies in the plane of the horizontal plates (e.g. the plating) shown in Figure 13 and where the connection of the perpendicular (web) plates is of secondary importance, welds uniting three plates may be made in accordance with Figure 13 (with the exception of those subjected mainly to dynamic loads). For the root passes of the three plate weld sufficient penetration shall be achieved. Sufficient penetration has to be verified in way of the welding procedure test.

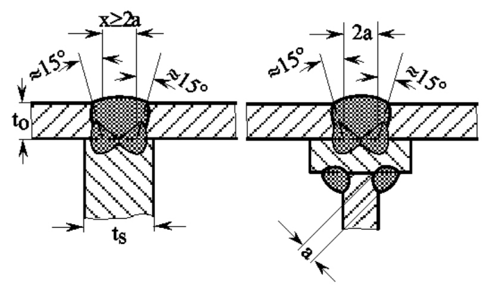


Figure 13 Welding together three plates

The effective thickness of the weld connecting the horizontal plates shall be determined in accordance with the second paragraph of this section. The requisite *a* dimension is determined by the joint uniting the vertical (web) plates and shall, where necessary, be determined in accordance with Table 3 or by calculation as for fillet welds.

The following table shows reference values for the design of three plate connections at rudders, steering nozzle, etc.

Plating thickness t_o [mm]	≤ 10	12	14	16	18	≥ 20
Max. weld gap x [mm]	6	7	8	10	11	12
Min. web thickness t_s [mm]	10	12	14	16	18	20

2.3.3 Fillet weld connections

In principle fillet welds shall be of the double fillet weld type. Exceptions to this rule (as in the case of closed box girders and mainly shear stresses parallel to the weld) are subject to approval in each individual case. The throat thickness *a* of the weld (the height of the inscribed isosceles triangle) shall be determined in accordance with Table 3 or by calculation according to C3. The leg length of a fillet weld shall be not less than 1,4 times the throat thickness *a*. For fillet welds at doubling plates, see [2.2.4]; for the welding of the deck stringer to the sheer strake, see Ch.4 Sec.3 [10.3] and for bracket joints, see [3.2.7].

The relative fillet weld throat thicknesses specified in Table 3 relate to normal and higher strength hull structural steels and comparable structural steels. They may also be generally applied to high strength structural steels and non-ferrous metals provided that the tensile shear strength of the weld metal used is at least equal to the tensile strength of the base material. Failing this, the *a* dimension shall be increased accordingly and the necessary increment shall be established during the welding procedure test (see Pt.2). Alternatively proof by calculation taking account of the properties of the weld metal may be presented.

Note:

In the case of higher-strength aluminium alloys (e.g. AlMg4,5Mn), such an increment may be necessary for cruciform joints subject to tensile stresses, as experience shows that in the welding procedure tests the tensile-shear strength of fillet welds (made with matching filler metal) often fails to attain the tensile strength of the base material. See also Pt.2.

---e-n-d---o-f---n-o-t-e---

The throat thickness of fillet welds shall not exceed 0.7 times the lesser thickness of the parts to be connected (generally the web thickness). The minimum throat thickness is defined by the expression:

$$a_{\min} = \sqrt{\frac{t_1 + t_2}{3}} \text{ [mm]}$$

but not less than 2.5 mm

t_1 = lesser (e.g. the web) plate thickness [mm]

t_2 = greater (e.g. the flange) plate thickness [mm].

A smaller minimum fillet weld throat thickness may be agreed to if its faultless execution is demonstrated by means of a welding procedure test.

It is desirable that the fillet weld section shall be flat faced with smooth transitions to the base material. Where proof of fatigue strength is required, see App.D, machining of the weld (grinding to remove notches) may be required depending on the notch category. The weld should penetrate at least close to the theoretical root point.

Where mechanical welding processes are used which ensure deeper penetration extending well beyond the theoretical root point and where such penetration is uniformly and dependably maintained under production conditions, approval may be given for this deeper penetration to be allowed for in determining the throat thickness. The effective dimension:

$$a_{deep} = a + \frac{2\min e}{3} \text{ [mm]}$$

shall be ascertained in accordance with Figure 14 and by applying the term min *e* to be established for each welding process by a welding procedure test. The throat thickness shall not be less than the minimum throat thickness related to the theoretical root point.

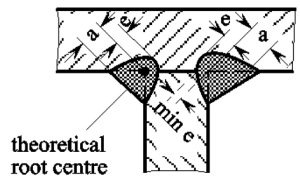


Figure 14 Fillet welds with increased penetration

When welding on top of shop primers which are particularly liable to cause porosity, an increase of the *a* dimension by up to 1 mm may be stipulated depending on the welding process used. This is specially applicable where minimum fillet weld throat thicknesses are employed. The size of the increase shall be decided on a case by case basis considering the nature and severity of the stressing following the test results of the shop primer in accordance with the Pt.2. This applies in analogous manner to welding processes where provision has to be made for inadequate root penetration.

Strengthened fillet welds continuous on both sides shall be used in areas subjected to severe dynamic loads (e.g. for connecting the longitudinal and transverse girders of the engine base to top plates close to foundation bolts, see Table 3), unless single or double-bevel welds are stipulated in these locations. In these areas the *a* dimension shall equal 0.7 times the lesser thickness of the parts to be welded.

Intermittent fillet welds in accordance with Table 3 may be located opposite one another (chain intermittent welds, possibly with scallops) or may be staggered (see Figure 15). In case of small sections other types of scallops may be accepted.

In water and cargo tanks, in the bottom area of fuel oil tanks and of spaces where condensed or sprayed water may accumulate and in hollow components (e.g. rudders) threatened by corrosion, only continuous or intermittent fillet welds with scallops shall be used. This applies accordingly also to areas, structures or spaces exposed to extreme environmental conditions or which are exposed to corrosive cargo.

There shall be no scallops in areas where the plating is subjected to severe local stresses (e.g. in the bottom section of the fore ship) and continuous welds shall be preferred where the loading is mainly dynamic.

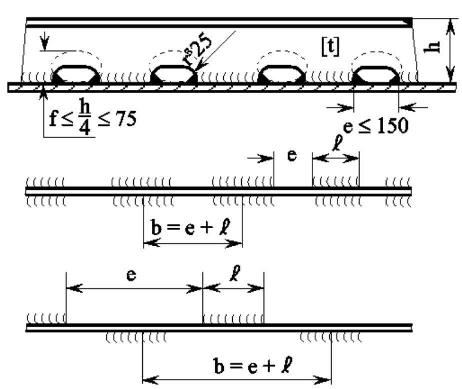


Figure 15 Scallop, chain and staggered welds

The throat thickness a_u of intermittent fillet welds shall be determined according to the selected pitch ratio b/ℓ by applying the equation:

$$a_u = 1.1 \cdot a\left(\frac{b}{\ell}\right)[\mathrm{mm}]$$

where:

a	=	required fillet weld throat thickness [mm] for a continuous weld according to Table 3 or determined by calculation
b	=	pitch [mm]
	=	$e + \ell$
e	=	interval between the welds [mm]
ℓ	=	length of fillet weld [mm].

The pitch ratio b/ℓ should not exceed 5. The maximum unwelded length ($b - \ell$ with scallop and chain welds, or $b/2 - \ell$ with staggered welds) should not exceed 25 times the lesser thickness of the parts to be welded. The length of scallops should, however, not exceed 150 mm.

Lap joints should be avoided wherever possible and shall not be used for heavily loaded components. In the case of components subject to low loads lap joints may be accepted provided that, wherever possible, they are orientated parallel to the direction of the main stress. The width of the lap shall be 1.5 t + 15 mm (t = thickness of the thinner plate). Except where another value is determined by calculation, the fillet weld throat thickness a shall equal 0.4 times the lesser plate thickness, subject to the requirement that it shall not be less than the minimum throat thickness required by the third paragraph of this section. The fillet weld shall be continuous on both sides and shall meet at the ends.

In the case of plug welding, the plugs should, wherever possible, take the form of elongated holes lying in the direction of the main stress. The distance between the holes and the length of the holes may be determined by analogy with the pitch b and the fillet weld length ℓ in the intermittent welds covered by a previous paragraph. The fillet weld throat thickness a_u may be established as classified before. The width of the holes shall be equal to at least twice the thickness of the plate and shall not be less than 15 mm. The ends of the holes shall be semi-circular. Plates or sections placed underneath should at least equal the perforated plate in thickness and should project on both sides to a distance of 1.5 × the plate thickness subject to a maximum of 20 mm. Wherever possible only the necessary fillet welds shall be welded, while the remaining void is packed with a suitable filler. Lug joint welding is not allowed.

2.4 Welded joints of particular components

2.4.1 Welds at the ends of girders and stiffeners

As shown in Figure 16, the web at the end of intermittently welded girders or stiffeners shall be continuously welded to the plating or the flange plate, as applicable, over a distance at least equal to the depth h of the girder or stiffener subject to a maximum of 300 mm. Regarding the strengthening of the welds at the ends, extending normally over 0.15 · ℓ of the span, see Table 3.

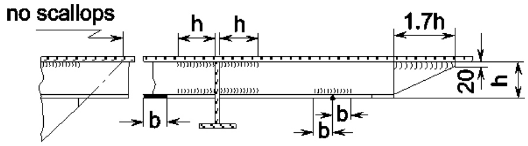


Figure 16 Welds at the ends of girders and stiffeners

The areas of bracket plates should be continuously welded over a distance at least equal to the length of the bracket plate. Scallops shall be located only beyond a line imagined as an extension of the free edge of the bracket plate.

Wherever possible, the free ends of stiffeners shall abut against the transverse plating or the webs of sections and girders so as to avoid stress concentrations in the plating. Failing this, the ends of the stiffeners shall be sniped and continuously welded over a distance of at least 1.7 *h* subject to a maximum of 300 mm.

Where butt joints occur in flange plates, the flange shall be continuously welded to the web on both sides of the joint over a distance at least equal to the width of the flange.

2.4.2 Joints between section ends and plates

Welded joints connecting section ends and plates may be made in the same plane or lapped.

Where no design calculations have been carried out or stipulated for the welded connections, the joints may be analogously to those shown in Figure 17.

Where the joint lies in the plane of the plate, it may conveniently take the form of a single-bevel butt weld with fillet. Where the joint between the plate and the section end overlaps, the fillet weld shall be continuous on both sides and shall meet at the ends. The necessary *a* dimension shall be calculated in accordance with [3.2.6]. The fillet weld throat thickness shall not be less than the minimum specified in [2.3.3].

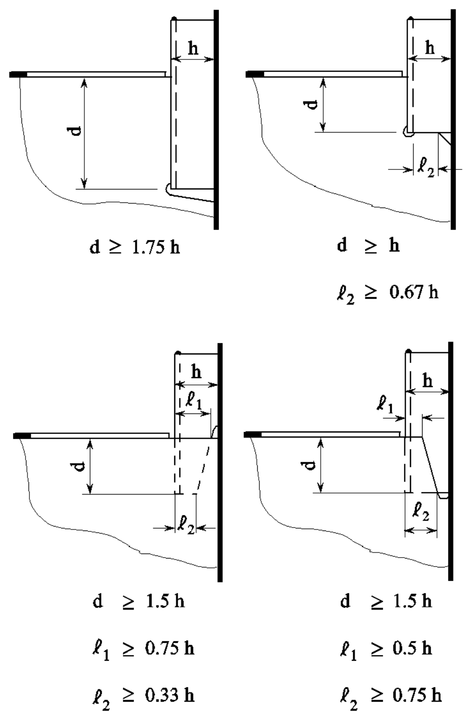


Figure 17 Joints uniting section ends and plates

2.4.3 Welded propeller bracket joints

Unless cast in one piece or provided with integrally cast welding flanges analogous to those prescribed in [2.2.1], see Figure 18, strut barrel and struts shall be connected to each other and to the shell plating in the manner shown in Figure 19.

In the case of single-strut shaft brackets no welding shall be performed on the arm or close to the position of constraint. Such components shall be provided with integrally forged or cast welding flanges.

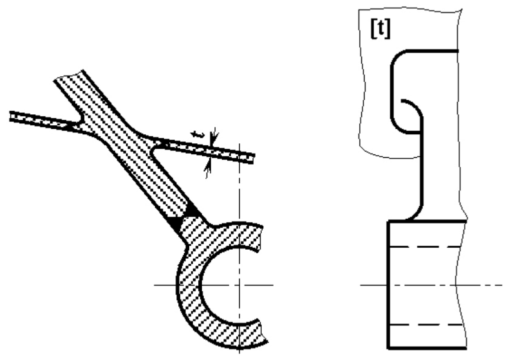
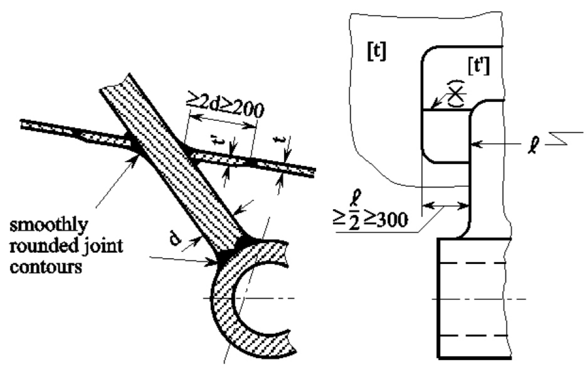


Figure 18 Propeller bracket with integrally cast welding flanges



t = plating thickness in accordance with Section 6, F. in [mm]

t' = $\frac{d}{3}$ +5 [mm] where d < 50mm

t' = 3 $\sqrt{d}$ [mm] where d ≥ 50mm

For shaft brackets of elliptically shaped cross section d may be substituted by 2/3 d in the above formulae.

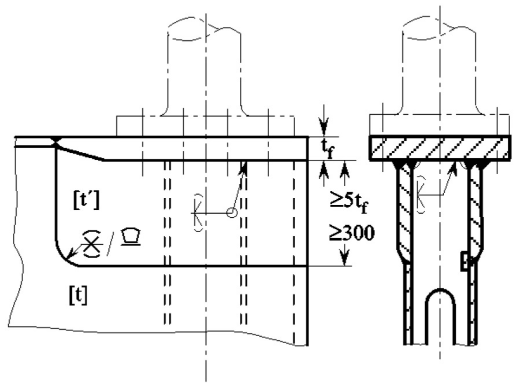
Figure 19 Shaft bracket without integrally cast welding flanges

2.4.4 Rudder coupling flanges

Unless forged or cast steel flanges with integrally forged or cast welding flanges in conformity with [2.2.1] are used, horizontal rudder coupling flanges shall be joined to the rudder body by plates of graduated thickness and full penetration single or double-bevel welds as prescribed in [2.3.2], see Figure 20.

Allowance shall be made for the reduced strength of the coupling flange in the thickness direction, see [2.1.5] and [2.2.5]. In case of doubt, proof by calculation of the adequacy of the welded connection shall be produced.

The welded joint between the rudder stock (with thickened collar, see [2.2.1]) and the flange shall be made in accordance with Figure 21.



t = plate thickness in accordance with Ch.7 Sec.2 [7.3.1] [mm]

t_f = actual flange thickness in [mm]

t' = $\frac{t_f}{3} + 5$ [mm] where $t_f < 50$ mm

t' = $3\sqrt{t_f}$ [mm] where $t_f \geq 50$ mm

Figure 20 Horizontal rudder coupling flanges

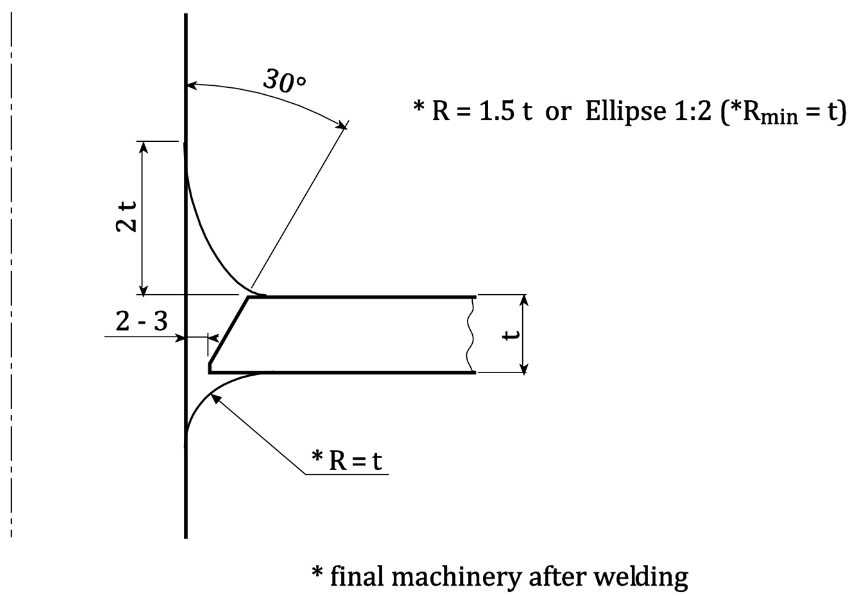


Figure 21 Welded joint between rudder stock and coupling flange

3 Stress analysis

3.1 General analysis of fillet weld stresses

3.1.1 Definition of stresses

For calculation purposes, the following stresses in a fillet weld are defined (see also Figure 22):

$\sigma_\perp$ = normal stresses acting vertically to the direction of the weld seam

$\tau_\perp$ = shear stress acting vertically to the direction of the weld seam

$\tau_{//}$ = shear stress acting in the direction of the weld seam.

Normal stresses acting in the direction of the weld seam need not be considered.

For calculation purposes the weld seam area is $a \cdot \ell$.

Due to equilibrium conditions the following applies to the flank area vertical to the shaded weld seam area:

$\tau_\perp = \sigma_\perp$

The equivalent stress shall be calculated by the following equation:

$$\sigma_v = \sqrt{\sigma_\perp^2 + \tau_\perp^2 + \tau_\parallel^2}$$

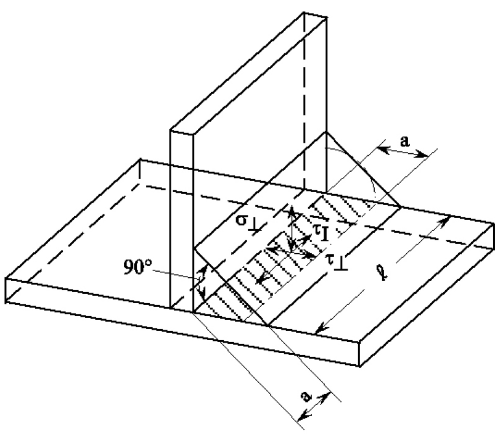


Figure 22 Stresses in a fillet weld

3.1.2 Definitions

a = throat thickness [mm]

ℓ = length of fillet weld [mm]

P = single force [N]

M = bending moment at the position considered [Nm]

Q = shear force at the point considered [N]

S = first moment of the cross sectional area of the flange connected by the weld to the web in relation to the neutral beam axis [cm^3]

I = moment of inertia of the girder section [cm^4]

W = section modulus of the connected section [cm^3].

3.2 Determination of stresses

3.2.1 Fillet welds stressed by normal and shear forces

$$\sigma = \tau = \frac{P}{\Sigma a \cdot \ell} \text{ [MPa]}$$

Flank an frontal welds are regarded as being equal for the purposes of stress analysis. In view of this, normal and shear stresses are calculated as follows:

Joint as shown in Figure 23:

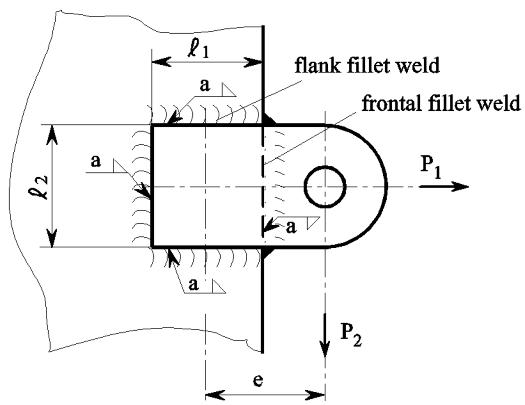


Figure 23 Weld joint of an overlapped lifting eye

— Stresses in frontal fillet welds:

$$\tau_\perp = \frac{P_1}{2 \cdot a \cdot (\ell_1 + \ell_2)} \text{ [MPa]}$$

$$\tau_\parallel = \frac{P_2}{2 \cdot a \cdot (\ell_1 + \ell_2)} \pm \frac{P_2 \cdot e}{2 \cdot a \cdot F_t} \text{ [MPa]}$$

$$F_t = (\ell_1 + a) \cdot (\ell_2 + a) \text{ [mm}^2\text{]}$$

— Stresses in flank fillet welds:

$$\tau_\perp = \frac{P_2}{2 \cdot a \cdot (\ell_1 + \ell_2)} \text{ [MPa]}$$

$$\tau_\parallel = \frac{P_1}{2 \cdot a \cdot (\ell_1 + \ell_2)} \pm \frac{P_2 \cdot e}{2 \cdot a \cdot F_t} \text{ [MPa]}$$

ℓ_1, ℓ_2, e [mm]

— Equivalent stress for frontal and flank fillet welds:

$$\sigma_v = \sqrt{\tau_\perp^2 + \tau_\parallel^2}$$

Joint as shown in Figure 24.

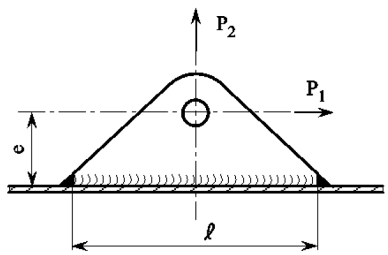


Figure 24 Weld joint of a vertically mounted lifting eye

$$\tau_{\perp} = \frac{P_2}{2 \cdot \ell \cdot a} + \frac{3 \cdot P_1 \cdot e}{\ell^2 \cdot a} \text{ [MPa]}$$

$$\tau_{\parallel} = \frac{P_1}{2 \cdot \ell \cdot a} \text{ [MPa]}$$

Equivalent stress:

$$\sigma_v = \sqrt{\tau_{\perp}^2 + \tau_{\parallel}^2}$$

3.2.2 Fillet weld joints stressed by bending moments and shear forces

The stresses at the fixing point of a girder are calculated as follows (in Figure 25 a cantilever beam is given as an example):

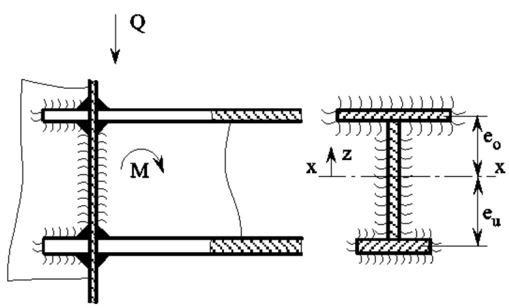


Figure 25 Fixing point of a cantilever beam

— Normal stress due to bending moment:

$$\sigma_{\perp}(z) = \frac{M}{I_s} \cdot z \text{ [MPa]}$$

$$\sigma_{\perp\ \max}(z) = \frac{M}{I_s} \cdot e_u \text{ [MPa], if } e_u > e_0$$

$$\sigma_{\perp\ \max}(z) = \frac{M}{I_s} \cdot e_0 \text{ [MPa], if } e_u < e_0$$

— Shear stress due to shear force:

$$\tau_{\parallel}(z) = \frac{Q \cdot S_s(z)}{10 \cdot I_s \cdot \Sigma a} \text{ [N/mm}^2\text{]}$$

$$\tau_{\parallel\ max} = \frac{Q \cdot S_{s\ max}}{20 \cdot I_s \cdot a}\ [\text{N/mm}^2]$$

where:

I_s = moment of inertia of the welded joint related to the x-axis [cm^4]
$S_s(z)$ = the first moment of the connected weld section at the point under consideration [cm^3]
Z = distance from the neutral axis [cm].

— Equivalent stress:

It has to be proved that neither $\sigma_{\perp\ max}$ in the region of the flange nor $\tau_{\parallel\ max}$ in the region of the neutral axis nor the equivalent stress $\sigma_v = \sqrt{\sigma_\perp^2 + \tau_\parallel^2}$ exceed the permitted limits given in [3.2.8] at any given point. The equivalent stress σ_v should always be calculated at the web-flange connection.

3.2.3 Fillet welded joints stressed by bending and torsional moments and shear forces

Regarding the normal and shear stresses resulting from bending, see [3.2.2]. Torsional stresses resulting from the torsional moment M_T shall be calculated:

$$\tau_T = \frac{M_T \cdot 10^3}{2 \cdot a \cdot A_m}\ [\text{N/mm}^2]$$

where:

M_T = torsional moment [Nm]
A_m = sectional area [mm^2] enclosed by the weld seam.

The equivalent stress composed of all three components (bending, shear and torsion) is calculated by means of the following equations:

$$\sigma_v = \sqrt{\sigma_\perp^2 + \tau_\parallel^2 + \tau_T^2}\ [\text{N/mm}^2]$$

where $\tau_\parallel$ and τ_T have not the same direction

$$\sigma_v = \sqrt{\sigma_\perp^2 + \left(\tau_\parallel + \tau_T\right)^2}\ [\text{N/mm}^2]$$

where $\tau_\parallel$ and τ_T have the same direction.

3.2.4 Continuous fillet welded joints between web and flange of bending girders

The stresses shall be calculated in way of maximum shear forces. Stresses in the weld's longitudinal direction need not to be considered.

In the case of continuous double fillet weld connections the shear stress shall be calculated as follows:

$$\tau_\parallel = \frac{Q \cdot S}{20 \cdot I \cdot a}\ [\text{N/mm}^2]$$

The fillet weld thickness required is:

$$a_{req} = \frac{Q \cdot S}{20 \cdot I \cdot \tau_{zul}}\ [\text{mm}]$$

3.2.5 Intermittent fillet welded joints between web and flange of bending girders

Shear stress:

$$\tau_{\parallel} = \frac{Q \cdot S \cdot \alpha}{20 \cdot I \cdot a}\left(\frac{b}{\ell}\right) \text{ [N/mm}^2\text{]}$$

where:

b = pitch

α = 1.1 stress concentration factor which takes into account increases in shear stress at the ends of the fillet weld seam ℓ.

The fillet weld thickness required is:

$$a_{req} = \frac{Q \cdot S \cdot 1.1}{20 \cdot I \cdot \tau_{zul}}\left(\frac{b}{\ell}\right) \text{ [mm]}$$

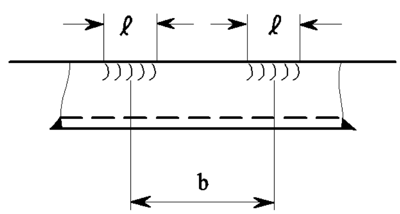


Figure 26 Intermittent fillet weld joints

3.2.6 Fillet weld connections on overlapped profile joints

Profiles joined by means of two flank fillet welds (see Figure 27):

$$\tau_{\perp} = \frac{Q}{2 \cdot a \cdot d} \text{ [N/mm}^2\text{]}$$

$$\tau_{\parallel} = \frac{M \cdot 10^3}{2 \cdot a \cdot c \cdot d} \text{ [N/mm}^2\text{]}$$

The equivalent stress is:

$$\sigma_v = \sqrt{\tau_{\perp}^2 + \tau_{\parallel}^2}$$

c, *d*, ℓ_1, ℓ_2, *r* [mm] see Figure 27

$$c = r + \frac{3\,\ell_1 - \ell_2}{4} \text{ [mm]}$$

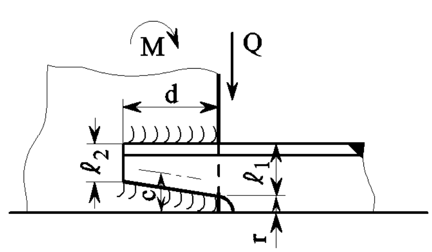


Figure 27 Profile joined by means of two flank fillet joints

As the influence of the shear force can generally be neglected, the required fillet weld thickness may be determined by the following equation:

$$a_{req} = \frac{W \cdot 10^3}{1.5 \cdot c \cdot d} [mm]$$

Profiles joined by means of two flank and two frontal fillet welds (all round welding as shown in Figure 28):

$$\tau_{\perp} = \frac{Q}{a \cdot \left(2d + \ell_1 + \ell_2\right)} \text{ [N/mm}^2\text{]}$$

$$\tau_{\parallel} = \frac{M \cdot 10^3}{a \cdot c \cdot \left(2d + \ell_1 + \ell_2\right)} \text{ [N/mm}^2\text{]}$$

The equivalent stress is:

$$\sigma_v = \sqrt{\tau_{\perp}^2 + \tau_{\parallel}^2}$$

$$a_{req} = \frac{W \cdot 10^3}{1.5 \cdot c \cdot d \left(1 + \frac{\ell_1 + \ell_2}{2\,d}\right)} [mm]$$

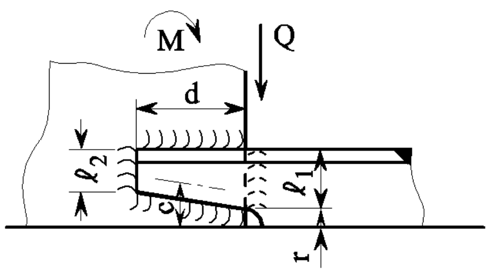


Figure 28 Profile joined by means of two flank and two frontal fillet welds (all round welding)

3.2.7 Bracket joints

Where profiles are joined to brackets as shown in Figure 29, the average shear stress is:

$$\tau = \frac{3 \cdot M \cdot 10^3}{4 \cdot a \cdot d^2} + \frac{Q}{2 \cdot a \cdot d} \text{ [N/mm}^2\text{]}$$

where:

d = length of overlap [mm].

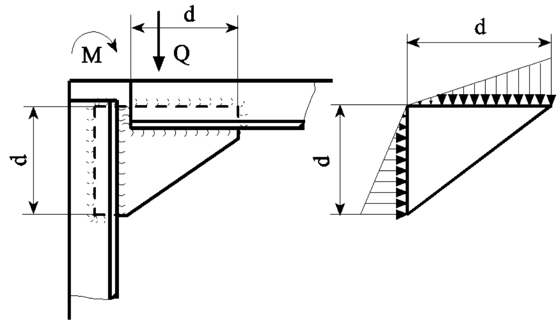


Figure 29 Bracket joint with idealized stress distribution resulting from moment *M* and shear force *Q*

The required fillet weld thickness shall be calculated from the section modulus of the profile as follows:

$$a_{req} = \frac{1000 \cdot W}{d^2} \text{ [mm]}$$

(The shear force Q has been neglected.)

3.2.8 Permissible stresses

The permissible stresses for various materials under mainly static loading conditions are given in Table 2. The values listed for high strength steels, austenitic stainless steels and aluminium alloys are based on the assumption that the strength values of the weld metal used are at least as high as those of the parent metal. If this is not the case, the *a*-value calculated shall be increased accordingly (see also [2.3.3]).

Table 2 Permissible stresses in fillet weld seams

Material		R_y *[MPa]*	*Permissible stresses for: equivalent stress* σ_{vp} *and shear stress* τ_p *[MPa]*
Normal strength hull structural	VL - A / B / D / E	235	115
Steel higher strength structural steel	VL - A / D / E / F 32	315	145
	VL - A / D / E / F 36	355	160
	VL - A / D / E / F 40	390	175
High strength steels	S 460	460	200
	S 690	685	290
Austenitic and austenitic-ferritic stainless steels	1.4306 / 304 L	180	110
	1.4404 / 316 L	190	
	1.4435 / 316 L	190	
	1.4438 / 317 L	195	
	1.4541 / 321	205	

Material		*R_y [MPa]*	*Permissible stresses for: equivalent stress σ_{vp} and shear stress τ_p [MPa]*
	1.4571 / 316 Ti	215	
	1.4406 / 316 LN	280	
	1.4429 / 316 LN	295	130
	1.4439 / 317 LN	285	
	1.4462 / 318 LN	480	205
Aluminium alloys	5083 / AlMg4,5Mn0,7	125[1]	56
	5754 / AlMg3	80[1]	36
	6060 / AlMgSi	65[2]	30
	6082 / AlSi1MgMn	115[2]	51
1) Plates, soft condition 2) Sections, cold hardened			

Table 3 Fillet weld connections

Structural parts to be connected	*Basic thickness of fillet welds a/t_0 [1] for double continuous fillet welds [2]*	*Intermittent fillet welds permissible [3]*
Bottom structures		
transverse and longitudinal girders to each other	0.35	x
to shell and inner bottom	0.2	x
centre girder to flat keel and inner bottom	0.4	
transverse and longitudinal girders and stiffeners including shell plating in way of bottom strengthening forward	0.3	
machinery space		
transverse and longitudinal girders to each other	0.35	
to shell and inner bottom	0.3	
inner bottom to shell	0.4	
sea chests: water side	0.5	
inside	0.3	
Machinery foundation		
longitudinal and transverse girders to each other and to the shell	0.4	
— to inner bottom and face plates	0.4	
— to top plates	0.50 [4]	

Structural parts to be connected	*Basic thickness of fillet welds a/t_0 [1] for double continuous fillet welds [2]*	*Intermittent fillet welds permissible [3]*
— in way of foundation bolts	0.70 [4]	
— to brackets and stiffeners	0.3	
longitudinal girders of thrust bearing to inner bottom	0.4	
Decks		
— to shell (general)	0.4	
deckstringer to sheerstrake (see also Sec.4 [1.3.2])	0.5	
Frames, stiffeners, beams etc.		
general	0.15	x
in peak tanks	0.3	x
bilge keel to shell	0.15	
Transverses, longitudinal and transverse girders		
general	0.15	x
within 0.15 of span from supports	0.25	
cantilevers	0.4	
pillars to decks	0.4	
Bulkheads, tank boundaries, walls of superstructures and deckhouses		
to decks, shell and walls	0.4	
Rudder		
plating to webs	0.25	x
Stem		
plating to webs	0.25	x

1) t_0 = thickness of the thinner plate.
2) In way of large shear forces larger throat thicknesses may be required on the bases of calculations according to [3].
3) For intermittent welding in spaces liable to corrosion B.3.3.8 shall be observed.
4) For plate thicknesses exceeding 15 mm single or double bevel butt joints with, full penetration or with defined incomplete root penetration according to Figure 9 to be applied.

APPENDIX D FATIGUE

1 General

The proof of sufficient fatigue strength, i e. the strength against crack initiation under dynamic loads during operation, is useful for judging and reducing the probability of crack initiation of structural members during the design stage.

Due to the randomness of the load process, the variance of material properties, fabrication factors and due to effects of ageing, crack initiation cannot be completely excluded during later operation. Therefore, periodical surveys are necessary.

Fatigue strength of welded metal connections is primarily a function of the weld category (FAT-class) and the quality of the welding performance including its post-treatment. Fatigue strength of welded details is considered more or less independent from the mechanical basic strength of the material. Thus, it is of great importance that the weld provides low stress concentrations and is of a high quality, so that the use of very high strength steel is beneficial.

This appendix has particular attention not only to hull structures but also for sailing yacht keels.

1.1 Definitions

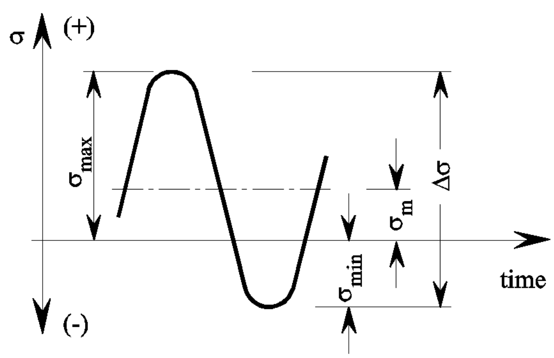


Figure 1 Definition of time-dependent stresses

$\Delta\sigma$	=	applied stress range (σ_{max} – σ_{min}), [MPa] see also Figure 1
σ_{max}	=	maximum upper stress of a stress cycle [MPa]
σ_{min}	=	maximum lower stress of a stress cycle [MPa]
$\Delta\sigma_{max}$	=	applied peak stress range within a stress range spectrum [MPa]
σ_m	=	mean stress ($\sigma_{max}/2 + \sigma_{min}/2$) [MPa]
$\Delta\sigma_p$	=	permissible stress range [MPa]
$\Delta\tau$	=	corresponding range for shear stress [MPa]
n	=	number of applied stress cycles
N	=	number of endured stress cycles according to S-N curve (= endured stress cycles under constant amplitude loading)
$\Delta\sigma_R$	=	fatigue strength reference value of S-N curve at $2\cdot10^6$ cycles of stress range [MPa] (= **FAT** class number according to Table 6)
f_m	=	correction factor for material effect
f_R	=	correction factor for mean stress effect
f_w	=	correction factor for weld shape effect

f_i = correction factor for importance of structural element
f_s = additional correction factor for structural stress analysis
f_t = correction factor for plate thickness effect
f_n = factor considering stress spectrum and number of cycles for calculation of permissible stress range
$\Delta\sigma_{Rc}$ = corrected fatigue strength reference value of S-N curve at $2 \cdot 10^6$ stress cycles [MPa]
D = cumulative damage ratio.

1.2 Scope

1.2.1 A fatigue strength analysis shall be performed for structures which are predominantly subjected to cyclic loads. Due consideration shall thereby be given to auxiliary structures such as e.g. fasteners. The notched details i.e. the welded joints as well as notches at free plate edges shall be considered individually.

The fatigue strength assessment shall be carried out either on the basis of a permissible peak stress range for standard stress spectra, see [2.2.1] or on the basis of a cumulative damage ratio, see [2.2.2].

For sailing yacht keels the fatigue strength assessment shall be carried out on the basis of a cumulative damage ratio, see [2.2.2].

1.2.2 No fatigue strength analysis is required if the peak stress range due to dynamic loads in the seaway (stress spectrum A according to [1.2.4]) and/or due to changing draught or loading conditions, respectively, fulfils the following conditions:

— peak stress range only due to seaway-induced dynamic loads:

$$\Delta\sigma_{\max} \leq 2.5\Delta\sigma_R$$

— sum of the peak stress ranges due to seaway-induced dynamic loads and due to changes of draught or loading condition, respectively:

$$\Delta\sigma_{\max} \leq 4.0\Delta\sigma_R$$

— for sailing yacht keels:

$$\Delta\sigma_{\max} \leq 2.5\Delta\sigma_R$$

where:

$\Delta\sigma_{\max} = 2 \cdot \Delta\sigma_n$

$\Delta\sigma_n$ from [2.2.5].

1.2.3 The rules are applicable to constructions made of normal and higher strength hull structural steels according to App.A [2] as well as of aluminium alloys according to App.A [4]. Other materials such as cast steel can be treated in an analogous manner by using appropriate design S-N curves.

Low cycle fatigue problems in connection with extensive cyclic yielding have to be specially considered. When applying the following rules, the calculated nominal stress range should not exceed 1.5 times the minimum yield stress.

1.2.4 The fatigue strength analysis is, depending on the detail considered, based on one of the following types of stress:

— For notches of free plate edges the notch stress σ_k, determined for linear-elastic material behaviour, is relevant, which can normally be calculated from a nominal stress σ_n and a theoretical stress concentration factor K_t. Values for K_t are given in App.A Figure 15 and App.A Figure 16 for different types of cut-outs. The fatigue strength is determined by the **FAT** class (or $\Delta\sigma_R$) according to Table 6, type E2 and E3.
— For welded joints the fatigue strength analysis is normally based on the nominal stress σ_n at the structural detail considered and on an appropriate detail classification as given in Table 6, which defines the **FAT** class (or $\Delta\sigma_R$).

— For those welded joints, for which the detail classification is not possible or additional stresses occur, which are not or not adequately considered by the detail classification, the fatigue strength analysis may be performed on the basis of the structural stress σ_s in accordance with [4].

1.3 Quality requirements (fabrication tolerances)

1.3.1 The detail classification of the different welded joints as given in Table 6 is based on the assumption that the fabrication of the structural detail or welded joint, respectively, corresponds in regard to external defects at least to quality group B according to DIN EN ISO 5817 and in regard to internal defects at least to quality group C. Further information about the tolerances can also be found in the rules for design, fabrication and inspection of welded joints.

A production standard which considers the special manufacturing requirements of yards has to be agreed case by case with the Society.

1.3.2 Relevant information has to be included in the manufacturing document for fabrication. If it is not possible to comply with the tolerances given in the standards, this has to be accounted for, when designing the structural details or welded joints, respectively. In special cases an improved manufacture as stated in [1.3.1] may be required, e.g. stricter tolerances or improved weld shapes, see also [3.3.2].

1.3.3 The following stress increase factors k_m for considering significant influence of axial and angular misalignment are already included in the fatigue strength reference values $\Delta\sigma_R$ (Table 6):

k_m = 1.15 butt welds (corresponding type A1, A2, A11)
= 1.30 butt welds (corresponding type A3–A10)
= 1.45 cruciform joints (corresponding type D1– D5)
= 1.25 fillet welds on one plate surface (corresponding type C7, C8).

Other additional stresses need to be considered separately.

2 Load spectrum/cycling regime

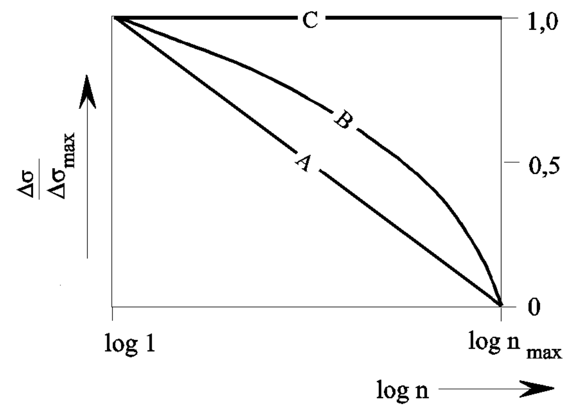
2.1 Seaway induced stress range for hull structures

2.1.1 The stress ranges $\Delta\sigma$ which shall be expected during the service life of the ship or structural component, respectively, may be described by a stress range spectrum (long-term distribution of stress range). Figure 2 shows three standard stress range spectra A, B and C, which differ from each other in regard to the distribution of stress range $\Delta\sigma$ as a function of the number of load cycles.

In general the fatigue analysis has to be performed for a number of cycles $n_{max} = 5 \cdot 10^7$ for seaway induced stresses with the stress range spectrum A. This considers a lifetime of 25 years with 230 days per year at sea in the North Atlantic.

Under extreme seaway conditions stress ranges exceeding $\Delta\sigma_{max}$ occur. These stress ranges, which load cycles shall be generally assumed with $n < 10^4$, can be neglected regarding the fatigue life, when the stress ranges $\Delta\sigma_{max}$ derived from loads according to Table 1 are assigned to the spectrum A.

Modified numbers of cyclic loads, load profiles and life time have to be agreed with the administration. For direct calculated stress range spectra (long term distributions of stress range), based on the requirements of Ch.3 Sec.3 [2.4], the factor f_Q in Table 1 has to be agreed with the Society.



A: Straight-line spectrum (typical stress range spectrum of seaway-induced stress ranges)

B: Parabolic spectrum (approximated normal distribution of stress range Δσ acc. DIN 15018)

C: Rectangular spectrum (constant stress range within the whole spectrum; typical spectrum of engine- or propeller-excited stress range)

Figure 2 Standard stress range spectra A, B and C

Table 1 Maximum and minimum value for variable cyclic loads

Load		Maximum load [1]		Minimum load [1]
Vertical and horizontal hull girder bending [2] (Ch.3, Sec.3, 2.)		$M_{SW} + f_Q \sqrt{M_{WV}^2 + M_{WH}^2}$		$M_{SW} - f_Q \sqrt{M_{WV}^2 + M_{WH}^2}$
Loads on weather decks, weather exposed walls and ship's shell (Ch.3, Sec.3, 3.2) [5]		$p_S = p_{Sstat} + p_{Sdyn}$		$p_s = p_{Sstat} - p_{Sdyn} \geq 0$
Liquid pressure in completely filled tanks (Ch.3, Sec.3, 3.3)	upright [5]	$p_{T1} = p_{T1stat} + p_{T1dyn}$		$p_{T1} = p_{T1stat} - p_{T1dyn}$
	heeled	$p_{T1} = \gamma_{fstat} \cdot p_{T1stat} + \gamma_{fdyn} \cdot \rho \cdot g (0,3 \cdot b + y) \sin\varphi$		$p_{T1} = \gamma_{fstat} \cdot p_{T1stat} + \gamma_{fdyn} \cdot \rho \cdot g (0,3 \cdot b - y) \sin\varphi$ but $\geq 100\ \Delta p$
Loads due to general stowage [3] (Ch.3, Sec.3, 3.4)		$p_L = p_{Lstat}(1 + a_z)$	vertical	$p_L = p_{Lstat}(1 - a_z)$
		$= p_{Lstat}\ a_x$	longitudinal	$= -p_{Lstat}\ a_x$
		$= p_{Lstat}\ a_y$	transverse	$= -p_{Lstat}\ a_y$
Loads due to rudder forces [4] (Ch.7, Sec.3, 2.1 and Sec.4, 1.2)		rudder force C_R rudder torque Q_R		$- C_R$ $- Q_R$

1 Maximum and minimum loads are to be determined that the largest applied stress range Δσ as per Fig. D.1 is obtained. The loads are to be superposed with the load combination factor ψ according to Ch.4, Sec.3 if applicable. For minor loads of the combination ψ may be reduced to 0,75.

2 For probability factor f_Q see Ch.3, Sec.3, for load collectives determined by direct calculation (see Ch.3, Sec. 3, 2.3.6.7) the probability factor f_Q has to be agreed with DNV GL

3 Probability factor $f_Q = 1,0$ is to be taken for determination of a_0 and further calculation of a_x and a_y .

4 In general the largest load is to be taken in connection with the load spectrum B according to Fig. D.2 without considering further cyclic loads.

5 Assumption of conservative superpositioning of sea and tank pressures within $0,2 < x/L \leq 0,7$: Where appropriate, proof is to be furnished for T_{min}.

The maximum and minimum stresses result from the maximum and minimum relevant seaway-induced load effects. The different load-effects for the calculation of $\Delta\sigma_{max}$ are, in general, to be superimposed conservatively

Table 1 shows examples for the individual loads which have to be considered in normal cases.

Other significant fluctuating stresses, e.g. in longitudinals due to deflections of supporting transverses as well as additional stresses due to the application of non-symmetrical sections, have to be considered, see Sec.3 Figure 1.

For ships of unconventional hull shape and for ships for which a special mission profile applies, a stress range spectrum deviating from spectrum A may be applied which may be evaluated by the spectral method.

2.1.2 Additional stress cycles resulting from changing mean stresses, e.g. due to changing tank loads or draught, need generally not be considered as long as the seaway-induced stress ranges are determined for the loading condition being most critical with respect to fatigue strength and the maximum change in mean stress is less than the maximum seaway-induced stress range.

Larger changes in mean stress shall be included in the stress range spectrum by conservative super positioning of the largest stress ranges, e.g. in accordance with the rain flow counting method.

2.2 Load spectrum/cycling for sailing yacht keels

2.2.1 General

From the dynamic motions of racing yachts, the dynamic stress amplitudes in a keel component have to be derived. These stress amplitudes vary with the characteristic motions of a yacht.

These rules are limited to evaluating only motions that excite the keel in transverse bending. This loading phenomenon is considered to highly contribute to fatigue-relevant loading. Other loading scenarios of a keel on the remaining 4 degrees of freedom should not be ignored but will not be addressed within these rules particularly.

The typical characteristic motions of keel transverse bending are divided in two groups:

— sailing (heeled) in seaways underlying vertical acceleration additional to gravity, see [2.2.2]
— change of tack, see [2.2.4].

Under this scope, racing yachts with keels fixed in centre-plane will be treated different from yachts with canting keels.

This cycling regime which shall be expected during the service life of a racing yacht keel will be described by a characteristic spectrum. This spectrum for racing yacht keels differs from common spectra used for ships in seaway condition or the ones used for more regular excitements like engine vibrations.

The characteristic cycling regime is intended to provide coverage of min. 60000 miles (depending on boat size) under random sea conditions. The dynamic loadings cumulating to this spectrum are derived from wave encounters resulting in amplified gravitational effects.

2.2.2 Design life/wave encounters

In order to establish the number of cycles and a mileage, a design life is defined including a percentage of this value being spent at sea.

Design life is an expression representing a theoretical time span in which fatigue degradation is not leading to premature failure, if the provisions of these guidelines are being followed.

A default value of this design life is 5 years with a fraction of 15% spend at sea, or the pertinent mileage.

Within these rules, the design life has a default consistency. It is divided in four characteristic headings, where a quarter of the life is absolved under each heading, see Table 2.

Table 2 Headings in life time; TWA = true wind angle

Heading	*of design lifetime*
Upwind 45° TWA	25%
Beam reach 90° TWA	25%
Broad reach 135° TWA	25%
Running 180° TWA	25%

Of this total time, the boat is expected to experience certain sea state conditions, defined by a wave length, each with a default share of the total time, as shown in Table 3.

Table 3 Sea condition relative to design life

Sea state	*of design lifetime*	*typical heel angle*	*wave length λ*
Extreme condition	1 %	45°	3 L_{WL}
Severe condition	5 %	35°	2 L_{WL}
Regular condition	10 %	30°	1 L_{WL}
Moderate condition	25 %	25°	0.75 L_{WL}
Light condition	59 %	15°	0.5 L_{WL}

For each sea state, the boat is dedicated a certain heel angle under sail, as per Table 3. For each combination heading/sea state, the encounter between boat and wave is determined using the following parameters:

— *Wave period:*
The wave periods used for this determination are the average wave periods for average wave lengths from *Wind and sea scale for fully arisen sea* from Myers et al. 1969. These average wave periods can be derived as a function of average wave length by using the following approximation:

$$T_{av} = 0.981 \cdot \sqrt{\lambda} \text{ [s]}$$

where:

λ = average wave length

— *Wave angular frequency:*

$$\omega = \frac{2 \cdot \pi}{T_{av}} \text{ [rad/s]}$$

The angular frequency of encounter between boat and wave:

$$\omega_e = \omega - \left(\frac{\omega^2}{9.81} \cdot \frac{v_0}{1.94} \cdot \cos(180 - \phi)\right)$$

where:

ϕ = heading or TWA [deg]
v_0 = boat design speed [kn].

The period of encounter between boat and wave:

$$T_e = \frac{2 \cdot \pi}{\omega_e} \text{ [s]}$$

The total number of cycles for each case heading φ/sea state can be determined by dividing the time spend by the period of encounter T_e.

2.2.3 Heave accelerations

In order to determine the pertinent heave accelerations of the boat for each of the case heading/sea state, the vertical acceleration of the wave surface is determined for each wave and amplified with a coefficient c_a as shown in Table 4. The latter shall estimate the typical response of boats on waves of a certain length.

The wave heights used for this determination are the significant wave heights from "Wind and sea scale for fully arisen sea" from Myers et al. 1969. These wave heights can be derived as a function of T_{av} by using the following approximation:

$$H_{av} = 0.0292 \cdot T_{av}^{2.43}$$

The vertical surface acceleration is determined as follows:

$$a_{vw} = \omega_e^2 \cdot \frac{H_{av}}{2} \text{ [m/s}^2\text{]}$$

Table 4 Amplifying coefficient c_a

Sea state	*Amplifying coefficient c_a*
Extreme	1.2
Severe	1.0
Regular	0.8
Moderate	0.6
Light	0.4

The vertical acceleration amplitude for the purpose of estimating gravitational loads on the keel is calculated as follows:

$$a_v = \frac{a_{vw}}{9.81} \cdot c_a \text{ [-]}$$

2.2.4 Design life - full reversal loads

The design life of a keel includes also full reversal loads. By nature these loads are different from the loads experienced by wave encounters as per [2.2.2], as they represent full reversal, occurring e.g. by the change of tack of a boat.

For the design life as per definition in [2.2.2], a number of 30 changes of tack per day are default, using the heel angles from Table 3 as full reversal amplitude.

2.2.5 Stress range

In order to transfer the cycling regime defined in [2.2] into a stress range spectrum required for the fatigue analysis, the pertinent stress ranges $\Delta\sigma_i$ can be calculated using the resulting keel angle to vertical and the resulting gravitational forces due to the defined accelerations and the basic static nominal stress calculated from load case 1 (90 degree heel), excluding c_d.

For cases sailing as per [2.2.2], the stress range values $\Delta\sigma_i$ are derived as follows:

$$\Delta\sigma_i = \sigma_n \cdot \sin\left(\alpha + \delta\right) \cdot 2 \cdot a_v$$

For cases knock down and tack/jibe as per [2.2.4], the stress range values $\Delta\sigma_i$ are derived as follows:

$$\Delta\sigma_i = \sigma_n \cdot \sin(\alpha + \delta) \cdot 2$$

where:

α = keel angle to vertical [deg]
δ = maximum canting angle to centre plane of yacht, for fixed keel yachts [deg]
σ_n = nominal stress from static load case LC1 at location of interest [MPa]
a_v = dynamic acceleration offset on gravity [g].

For each stress range $\Delta\sigma_i$ a value N_i will be determined (ultimate number of cycles) according to a standard Wöhler curve. The ratio between the defined cycles n_i and the ultimate cycles N_i will be called partial fatigue damage. Cumulating this ratio across the spectrum results in the overall fatigue damage ratio as defined in [3.3.2].

2.2.6 Combined stresses

The fatigue strength evaluation needs to be carried out at all locations suspect to high stressing.

Due to keel bending, not only tensile/compressive stresses occur, but also those combined with shear stresses, e.g. near the outboard junction of a shear web and plating.

In those cases, the equivalent stress shall be determined and the resulting value is subject to fatigue assessment, see [3.1.3].

For locations with a governing shear stress, alternative FAT class values shall be adopted as per definition in [3.1.5].

3 Fatigue strength analysis for welded joints using detail classification

3.1 Definition of nominal stress and detail classification for welded joints

3.1.1 Corresponding to their notch effect, welded joints are normally classified into detail categories considering particulars in geometry and fabrication, including subsequent quality control, and definition of nominal stress. Table 3 shows the detail classification based on recommendations of the International Institute of Welding (IIW) giving the **FAT** class ($\Delta\sigma_R$) for structures made of steel or aluminium alloys (Al).

In Table 4 $\Delta\sigma_R$-values for steel are given for some intersections of longitudinal frames of different shape and webs, which can be used for the assessment of the longitudinal stresses.

It has to be noted that some influence parameters cannot be considered by the detail classification and that a large scatter of fatigue strength has therefore to be expected.

3.1.2 Details which are not covered by Table 3 may be classified either on the basis of local stresses in accordance with [4]. or else, by reference to published experimental work or by carrying out special fatigue tests, assuming a sufficiently high confidence level, see [3.3.1] and taking into account the correction factors as given in [3.3].

3.1.3 Regarding the definition of nominal stress, the arrows in Table 3 indicate the location and direction of the stress for which the stress range shall be calculated. The potential crack location is also shown in Table 3. Depending on this crack location, the nominal stress range has to be determined by using either the cross sectional area of the parent metal or the weld throat thickness, respectively. Bending stresses in plate and shell structures have to be incorporated into the nominal stress, taking the nominal bending stress acting at the location of crack initiation.

Guidance note:

The factor K_s for the stress increase at transverse butt welds between plates of different thickness, see type 5 in Table 3 can be estimated in a first approximation as follows:

$$K_s = \frac{t_2}{t_1}$$

where:

t_1 = smaller plate thickness

t_2 = larger plate thickness.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

Additional stress concentrations which are not characteristic of the **FAT** class itself, e.g. due to cut-outs in the neighbourhood of the detail, have also to be incorporated into the nominal stress.

3.1.4 In the case of combined normal and shear stress the relevant stress range shall be taken as the range of the principal stress at the potential crack location which acts approximately perpendicular (within ±45°) to the crack front as shown in Table 3, as long as it is larger than the individual stress components.

3.1.5 Where solely shear stresses are acting, the largest principal stress $\sigma_1 = \tau$ may be used in combination with the relevant **FAT** class.

3.2 Permissible stress range for standard stress range spectra or calculation of the cumulative damage ratio

3.2.1 For standard stress range spectra according to Figure 2, the permissible peak stress range can be calculated as follows:

$$\Delta\sigma_p = f_n \cdot \Delta\sigma_{Rc}$$

where:

$\Delta\sigma_{Rc}$ = **FAT** class or fatigue strength reference value, respectively, corrected according to [3.3.2]

f_n = factor as given in Table 2.

The peak stress range of the spectrum shall not exceed the permissible value, i.e.

$$\Delta\sigma_{max} \le \Delta\sigma_p$$

3.2.2 If the fatigue strength analysis is based on the calculation of the cumulative damage ratio, the stress range spectrum expected during the envisaged service life shall be established, see [2.2.1] and the cumulative damage ratio D shall be calculated as follows:

$$D = \sum_{i=1}^{I} \left(n_i / N_i\right)$$

where:

I = total number of blocks of the stress range spectrum for summation (normally I > 20)

n_i = number of stress cycles in block i

N_i = number of endured stress cycles determined from the corrected design S-N curve (see [3.2]) taking $\Delta\sigma = \Delta\sigma_i$

$\Delta\sigma_i$ = stress range of block i.

To achieve an acceptable high fatigue life, the cumulative damage ratio should not exceed D = 1.

If the expected stress range spectrum can be superimposed by two or more standard stress spectra according to [1.2.4], the partial damage ratios D_i due to the individual stress range spectra can be derived from Table 2. In this case a linear relationship between number of load cycles and cumulative damage ratio may be assumed. The numbers of load cycles given in Table 2 apply for a cumulative damage ratio of D = 1.

3.3 Design S-N curves

3.3.1 Description of the design S-N curves

The design S-N curves for the calculation of the cumulative damage ratio according to [3.2.2] are shown in Figure 3 for welded joints at steel and in Figure 4 for notches at plate edges of steel plates.

For aluminium alloys (Al) corresponding S-N curves apply with reduced reference values of the S-N curve (**FAT** class) according to Table 6.

The S-N curves represent the lower limit of the scatter band of 95% of all test results available (corresponding to 97.5% survival probability) considering further detrimental effects in large structures.

To account for different influence factors, the design S–N curves have to be corrected according to [3.3.2].

Table 5 Factor f_n for the determination of the permissible stress range for standard stress range spectra

Stress range spectrum	Welded Joints ($m_o = 3$) n_{max} =					Plates Edges: Type E1 ($m_o = 5$) n_{max} =					Plates Edges: Type E2, E2a ($m_o = 4$) n_{max} =					Plates Edges: Type E3 ($m_o = 3{,}5$) n_{max} =				
	10^3	10^5	$5 \cdot 10^7$	10^8	$3 \cdot 10^8$	10^3	10^5	$5 \cdot 10^7$	10^8	$3 \cdot 10^8$	10^3	10^5	$5 \cdot 10^7$	10^8	$3 \cdot 10^8$	10^3	10^5	$5 \cdot 10^7$	10^8	$3 \cdot 10^8$
A		(17,2)	3,53	3,02	2,39		(8,1)	3,63	3,32	2,89		(8,63) (9,20)[3]	3,66	3,28	2,76		(10,3) (12,2)[2]	3,65	3,19	2,62
B		(9,2)	1,67	1,43	1,15	(9,5)	5,0	1,95	1,78	1,55	(10,30) (11,20)[3]	5,50 5,90[3]	1,86	1,65	1,40		6,6 7,5[2]	1,78	1,55	1,28
C	(12,6)	2,71	0,424 0,543[1]	0,369 0,526[1]	0,296 0,501[1]	(4,57)	1,82	0,606 0,673[1]	0,561 0,653[1]	0,500 0,621[1]	(4,57)	1,82	0,532 0,621[1]	0,482 0,602[1]	0,411 0,573[1]	(4,57)	1,82	0,483 0,587[1]	0,430 0,569[1]	0,358 0,541[1]

For definition of type E1 to type E3 see Table D.3

For definition of m_o see D.2.3.1.2

The values given in parentheses may be applied for interpolation.

For interpolation between any pair of values (n_{max1}; f_{n1}) and (n_{max2}; f_{n2}), the following formula may be applied in the case of stress spectrum A or B:

$$\log f_n = \log f_{n1} + \log\left(n_{max}/n_{max\,1}\right) \frac{\log\left(f_{n\,2}/f_{n1}\right)}{\log\left(n_{max\,2}/n_{max\,1}\right)}$$

For the stress spectrum C intermediate values may be calculated according to 2.3.1.2 by taking $N = n_{max}$ and $f_n = \Delta\sigma/\Delta\sigma_R$.

[1] f_n for non-corrosive environment, see also D.2.3.1.4.

[2] for $\Delta\sigma_R = 100$ [N/mm²]

[3] for $\Delta\sigma_R = 140$ [N/mm²]

The S-N curves represent section-wise linear relationships between log (Δσ) and log (N):

$$\log(N) = 7.0 + m \cdot Q$$

where:

Q = $\log(\Delta\sigma_R/\Delta\sigma) - 0.69897/m_0$

m = slope exponent of S-N curve, see [3.3.1] and [3.3.2]

m_0 = slope exponent in the range $N \leq 1 \cdot 10^7$

= 3 for welded joints

= 3.5 ÷ 5 for free plate edges, see Figure 4.

The S-N curve for **FAT** class 160 forms the upper limit for the S-N curves of free edges of steel plates with detail categories 100 – 150 in the range of low stress cycles, see Figure 4. The same applies accordingly to **FAT** classes 32 to 40 of aluminium alloys with an upper limit of **FAT** 71, see type E1 in Table 6.

For structures subjected to variable stress ranges, the S-N curves shown by the solid lines in Figure 3 and Figure 4 have to be applied (S-N curves of type M), i.e.

$m = m_0$ for $N \leq 10^7$ $(Q \leq 0)$

$m = 2 \cdot m_0 - 1$ for $N > 10^7$ $(Q > 0)$

For stress ranges of constant magnitude (stress range spectrum C) in non-corrosive environment from $N = 1 \cdot 10^7$ the S-N curves of type O in Figure 3 and Figure 4 can be used, thus:

$m = m_0$ for $N \leq 10^7$ $(Q \leq 0)$

$m = 22$ for $N > 10^7$ $(Q > 0)$

3.3.2 Correction of the reference value of the design S-N curve

A correction of the reference value of the S-N curve (or FAT class) is required to account for additional influence factors on fatigue strength as follows:

$$\Delta\sigma_{Rc} = f_m \, f_R \, f_W \, f_i \, f_t \, \Delta\sigma_R$$

f_m, f_R, f_w, f_i, f_t are defined in following sections.

For the description of the corrected design S-N curve, the equations given in [3.3.1] may be used by replacing $\Delta\sigma_R$ by $\Delta\sigma_{Rc}$.

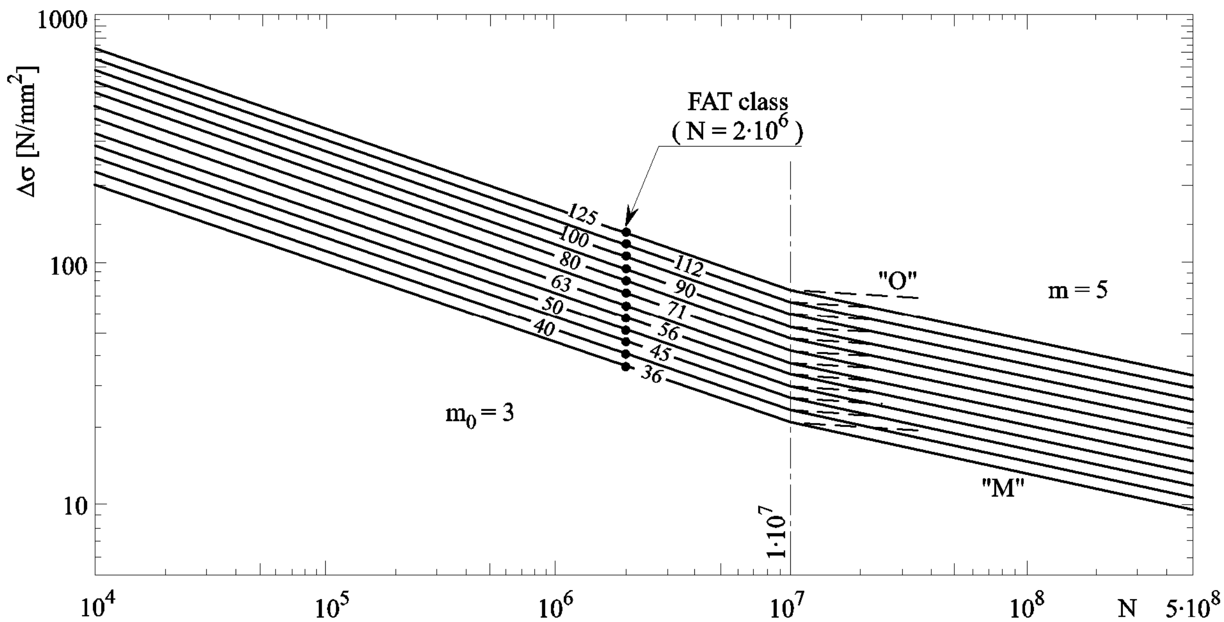


Figure 3 S-N curves for welded joints at steel

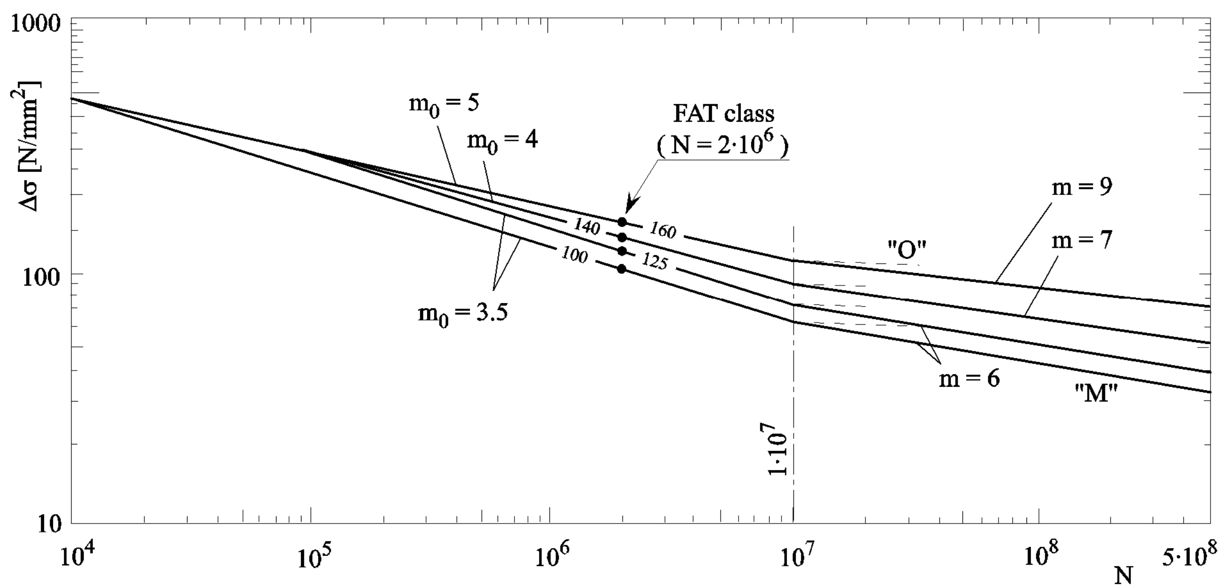


Figure 4 S-N curves for notches at plate edges of steel plates

Material effect (f_m)

For welded joints it is generally assumed that the fatigue strength is independent of steel strength, i.e.:

f_m = 1.0

For free edges at steel plates the effect of the material's yield point is accounted for as follows:

$$f_m = 1 + \frac{R_y - 235}{1200}$$

where R_{eH} shall be taken into account not greater than 390 MPa for sailing yacht keels.

Guidance note:

The limitation originates from the use of ship building steels with yield strength of 390 MPa and below. Without further proof, this limit applies also for higher strength steels used for racing yacht keel construction.

This correction is leading to over-optimistic life cycle predictions when used with yield strengths exceeding this value. Should higher strength steels be used, either the limit of 390 MPa shall be used or for a more individual evaluation fatigue life data shall be supplied for the relevant material.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

For aluminium alloys, f_m = 1 generally applies.

Effect of mean stress (f_R)

The correction factor is calculated as follows:

— for sailing yacht keels, where in a conservative approach, the stress cycling is considered mainly to occur as pulsating tensile stress f_R = 1.0
— in the range of tensile pulsating stresses

$$\sigma_m \geq \frac{\Delta\sigma_{\max}}{2}$$

f_R = 1.0

— in the range of alternating stresses

$$-\frac{\Delta\sigma_{\max}}{2} \leq \sigma_m \leq \frac{\Delta\sigma_{\max}}{2}$$

f_R = $1 + c\left(1 - \frac{2 \cdot \sigma_m}{\Delta\sigma_{\max}}\right)$

— in the range of compressive pulsating stresses

$$\sigma_m \leq -\frac{\Delta\sigma_{\max}}{2}$$

f_R = $1 + 2 \cdot c$

c = 0 for welded joints subjected to constant stress cycles (stress range spectrum C)
= 0.15 for welded joints subjected to variable stress cycles (corresponding to stress range spectrum A or B)
= 0.3 for unwelded base material.

Effect of weld shape (f_w)

In normal cases:

f_w = 1.0

A factor f_w > 1.0 applies for welds treated e.g. by grinding. By this surface defects such as slag inclusions, porosity and crack-like undercuts shall be removed and a smooth transition from the weld to the base material shall be achieved. Final grinding shall be performed transversely to the weld direction. The depth should be approximately 0.5 mm larger than that of visible undercuts.

For ground weld toes of fillet and K-butt welds machined by:

— disc grinder: f_w = 1.15
— burr grinder: f_w = 1.30

Premise for this is that root and internal failures can be excluded. Application of toe grinding to improve fatigue strength is limited to following details of Table 6:

— butt welds of type A2, A3 and A5 if they are ground from both sides
— non-load-carrying attachments of type C1, C2, C5 and C6 if they are completed with a full penetration weld
— transverse stiffeners of type C7
— doubling plates of type C9 if the weld throat thickness according to App.C was increased by 30%
— cruciform and T-joints of type D1 with full penetration welds.

The corrected **FAT** class that can be reached by toe grinding is limited for all types of welded connections of steel to $f_w - \Delta\sigma_R$ = 100 MPa and of aluminium to $f_w - \Delta\sigma_R$ = 40 MPa.

For butt welds ground flush the corresponding **FAT** class has to be chosen, e.g. type A1, A10 or A12 in Table 6.

For endings of stiffeners or brackets, e.g. type C2 in Table 6, which have a full penetration weld and are completely ground flush to achieve a notch-free transition, the following factor applies:

$f_w = 1.4$

The assessment of a local post-weld treatment of the weld surface and the weld toe by other methods has to be agreed on in each case.

Influence of importance of structural element (f_i)

In general the following applies:

$f_i = 1.0$

For sailing yacht keel structural elements, the correction factor f_i shall be taken as:

$f_i = 0.9$

For secondary structural elements failure of which may cause failure of larger structural areas, the correction factor f_i shall be taken as:

$f_i = 0.9$

For notches at plate edges in general the following correction factor shall be taken which takes into account the radius of rounding:

$f_i = 0.9 + 5/r \leq 1.0$

r = notch radius [mm]; for elliptical roundings the mean value of the two main half axes may be taken.

Plate thickness effect

In order to account for the plate thickness effect, application of the reduction factor f_t is required by the Society for butt welds oriented transversely to the direction of applied stress for plate thicknesses t > 25 mm.

$$f_t = \left(\frac{25}{t}\right)^n$$

where:

n = 0.17 as welded
= 0.10 toe-ground.

For all other weld connections consideration of the thickness effect may be required subject to agreement with the Society.

4 Fatigue strength analysis for welded joints based on local stresses

4.1

Alternatively to the procedure described in the preceding, the fatigue strength analysis for welded joints may be performed on the basis of local stresses. For common plate and shell structures in ships the assessment based on the so-called structural (or hot-spot) stress σ_s is normally sufficient.

The structural stress is defined as the stress being extrapolated to the weld toe excluding the local stress concentration in the local vicinity of the weld, see Figure 5.

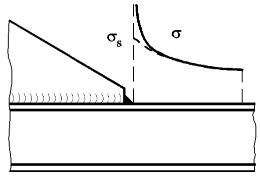


Figure 5 Structural stress

4.2

The structural stress can be determined by measurements or numerically, e.g. by the finite element method using shell or volumetric models, under the assumption of linear stress distribution over the plate thickness. Normally the stress is extrapolated linearly to the weld toe over two reference points which are located 0.5 and 1.5 × plate thickness away from the weld toe. In some cases the structural stress can be calculated from the nominal stress σ_n and a structural stress concentration factor K_s, which has been derived from parametric investigations using the methods mentioned. Parametric equations should be used with due consideration of their inherent limitations and accuracy.

4.3

For the fatigue strength analysis based on structural stress, the S-N curves shown in Figure 3 apply with the following reference values:

$\Delta\sigma_R$ = 100 (resp. 40 for Al):
for the butt welds types A1 – A6 and for K-butt welds with fillet welded ends, e.g. type D1 in Table 6, and for fillet welds which carry no load or only part of the load of the attached plate, type C1- C9 in Table 6

$\Delta\sigma_R$ = 90 (resp. 36 for Al):
for fillet welds, which carry the total load of the attached plate, e.g. type D2 in Table 6.

In special cases, where e.g. the structural stresses are obtained by non-linear extrapolation to the weld toe and where they contain a high bending portion, increased reference values of up to 15% can be allowed.

4.4

The reference value $\Delta\sigma_{Rc}$ of the corrected S–N curve shall be determined according to [3.3.2], taking into account the following additional correction factor which describes influencing parameters not included in the calculation model such as e.g. misalignment:

$$f_s = \frac{1}{k'_m - \frac{\Delta\sigma_{s,b}}{\Delta\sigma_{s,\max}}\left(k'_m - 1\right)}$$

where:

$\Delta\sigma_{s,max}$ = applied peak stress range within a stress range spectrum
$\Delta\sigma_{s,b}$ = bending portion of $\Delta\sigma_{s,max}$
k'_m = k_m – 0.05

k_m = stress increase factor due to misalignments under axial loading, at least k_m according to [1.3.3].

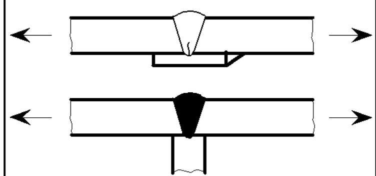
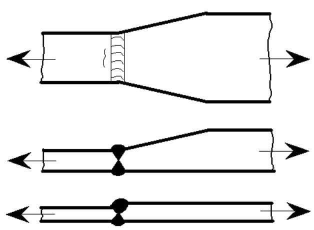
The permissible stress range or cumulative damage ratio, respectively, has to be determined according to [3.2].

4.5

In addition to the assessment of the structural stress at the weld toe, the fatigue strength with regard to root failure has to be considered by analogous application of the respective FAT class, e.g. type D3 of Table 6.

In this case the relevant stress is the stress in the weld throat caused by the axial stress in the plate perpendicular to the weld. It shall be converted at a ratio of $t/2a$.

Table 6 Catalogue of details

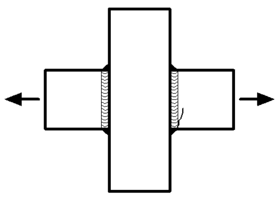
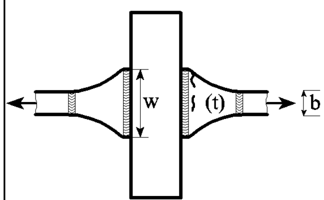
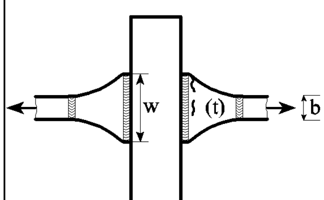
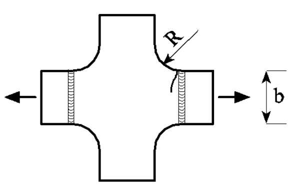
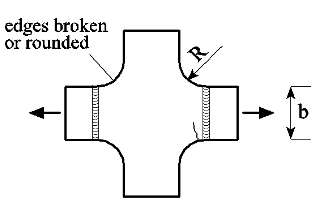
A. Butt welds, transverse loaded				
Type No.	Joint configuration showing mode of fatigue cracking and stress σ considered	Description of joint	FAT class $\Delta\sigma_R$	
			Steel	Al
A1		Transverse butt weld ground flush to plate, 100 % NDT (Non-Destructive Testing)	112	45
A2		Transverse butt weld made in shop in flat position, max. weld reinforcement 1 mm + 0,1 × weld width, smooth transitions, NDT	90	36
A3		Transverse butt weld not satisfying conditions for joint type No. A2, NDT	80	32
A4		Transverse butt weld on backing strip or three-plate connection with unloaded branch Butt weld, welded on ceramic backing, root crack	71 80	25 28
A5		Transverse butt welds between plates of different widths or thickness, NDT as for joint type No. A2, slope 1 : 5 as for joint type No. A2, slope 1 : 3 as for joint type No. A2, slope 1 : 2 as for joint type No. A3, slope 1 : 5 as for joint type No. A3, slope 1 : 3 as for joint type No. A3, slope 1 : 2 For the third sketched case the slope results from the ratio of the difference in plate thicknesses to the breadth of the welded seam. Additional bending stress due to thickness change to be considered, see also B.1.3.	 90 80 71 80 71 63	 32 28 25 25 22 20
A6		Transverse butt welds welded from one side without backing bar, full penetration root controlled by NDT not NDT For tubular profiles $\Delta\sigma_R$ may be lifted to the next higher FAT class.	 71 36	 28 12
A7		Partial penetration butt weld; the stress is to be related to the weld throat sectional area, weld overfill not to be taken into account	36	12

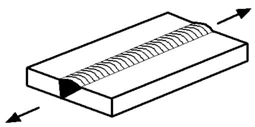
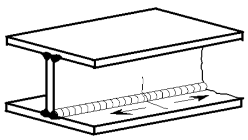
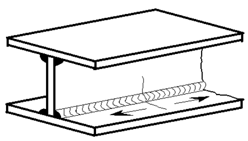
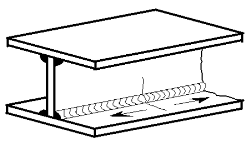
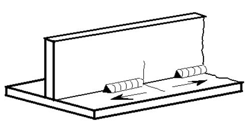
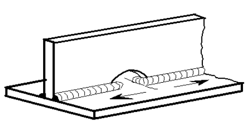
Note:

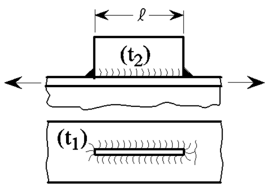
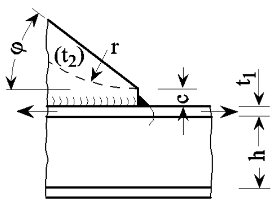
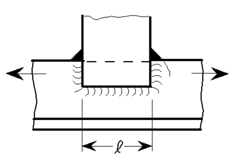
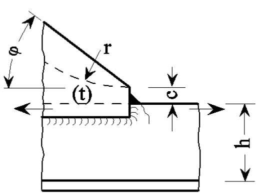
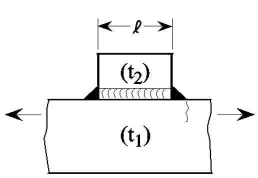
Correction to type No. A5

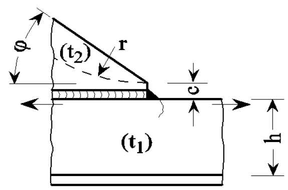
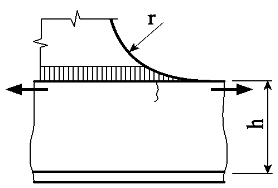
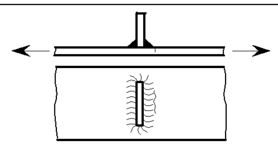
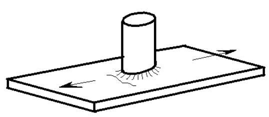
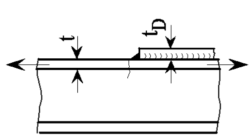
Additional bending stress due to thickness change to be considered, see also [3.1.3].

---e-n-d---o-f---n-o-t-e---

A. Butt welds, transverse loaded				
Type No.	**Joint configuration showing mode of fatigue cracking and stress σ considered**	**Description of joint**	**FAT class $\Delta\sigma_R$**	
			Steel	**Al**
A8		Full penetration butt weld at crossing flanges Welded from both sides.	50	18
A9		Full penetration butt weld at crossing flanges Welded from both sides Cutting edges in the quality according to type E2 or E3 Connection length w ≥ 2b $\sigma_{nominal} = \frac{F}{b \cdot t}$	63	22
A10		Full penetration butt weld at crossing flanges Welded from both sides, NDT, weld ends ground, butt weld ground flush to surface Cutting edges in the quality according to type E2 or E3 with $\Delta\sigma_R = 125$ Connection length w ≥ 2b $\sigma_{nominal} = \frac{F}{b \cdot t}$	80	32
A11		Full penetration butt weld at crossing flanges welded from both sides made in shop at flat position, radius transition with R ≥ b Weld reinforcement ≤ 1 mm + 0,1 x weld width, smooth transitions, NDT, weld ends ground Cutting edges in the quality according to type E2 or E3 with $\Delta\sigma_R = 125$	90	36
A12		Full penetration butt weld at crossing flanges, radius transition with R ≥ b Welded from both sides, no misalignment, 100 % NDT, weld ends ground, butt weld ground flush to surface Cutting edges broken or rounded according to type E2	100	40

B. Longitudinal load-carrying weld				
Type No.	Joint configuration showing mode of fatigue cracking and stress σ considered	Description of joint	FAT class $\Delta\sigma_R$	
			Steel	Al
B1		Longitudinal butt welds both sides ground flush parallel to load direction without start/stop positions, NDT with start/stop positions	 125 125 90	 50 50 36
B2		Continuous automatic longitudinal fully penetrated K-butt without stop/start positions (based on stress range in flange adjacent to weld)	125	50
B3		Continuous automatic longitudinal fillet weld penetrated K-butt weld without stop/start positions (based on stress range in flange adjacent to weld)	100	40
B4		Continuous manual longitudinal fillet or butt weld (based on stress range in flange adjacent to weld)	90	36
B5		Intermittent longitudinal fillet weld (based on stress range in flange at weld ends) In presence of shear τ in the web, the FAT class has to be reduced by the factor $(1 - \Delta\tau / \Delta\sigma)$, but not below 36 (steel) or 14 (Al).	80	32
B6		Longitudinal butt weld, fillet weld or intermittent fillet weld with cut outs (based on stress range in flange at weld ends) If cut out is higher than 40 % of web height In presence of shear τ in the web, the FAT class has to be reduced by the factor $(1 - \Delta\tau / \Delta\sigma)$, but not below 36 (steel) or 14 (Al). ***Note*** *For Ω-shaped scallops, an assessment based on local stresses in recommended.*	71 63	28 25

C. Non-load-carrying attachments				
Type No.	**Joint configuration showing mode of fatigue cracking and stress σ considered**	**Description of joint**	**FAT class $\Delta\sigma_R$**	
			Steel	**Al**
C1		Longitudinal gusset welded on beam flange, bulb or plate: $\ell \le 50$ mm 50 mm $< \ell \le 150$ mm 150 mm $< \ell \le 300$ mm $\ell > 300$ mm For $t_2 \le 0{,}5\ t_1$, $\Delta\sigma_R$ may be increased by one class, but not over 80 (steel) or 28 (Al); not valid for bulb profiles. When welding close to edges of plates or profiles (distance less than 10 mm) and/or the structural element is subjected to bending, $\Delta\sigma_R$ is to be decreased by one class.	 80 71 63 56	 28 25 20 18
C2		Gusset with smooth transition (sniped end or radius) welded on beam flange, bulb or plate; $c \le 2\ t_2$, max. 25 mm $r \ge 0{,}5$ h $r < 0{,}5$ h or $\varphi \le 20°$ $\varphi > 20°$ see joint type C1 For $t_2 \le 0{,}5\ t_1$, $\Delta\sigma_R$ may be increased by one class; not valid for bulb profiles. When welding close to the edges of plates or profiles (distance less than 10 mm), $\Delta\sigma_R$ is to be decreased by one class.	 71 63	 25 20
C3		Fillet welded non-load-carrying lap joint welded to longitudinally stressed component. – flat bar – to bulb section – to angle section For $\ell > 150$ mm, $\Delta\sigma_R$ has to be decreased by one class, while for $\ell \le 50$ mm, $\Delta\sigma_R$ may be increased by one class. If the component is subjected to bending, $\Delta\sigma_R$ has to be reduced by one class.	 56 56 50	 20 20 18
C4		Fillet welded lap joint with smooth transition (sniped end with $\varphi \le 20°$ or radius) welded to longitudinally stressed component. – flat bar – to bulb section – to angle section $c \le 2$ t, max. 25 mm	 56 56 50	 20 20 18
C5		Longitudinal flat side gusset welded on plate or beam flange edge $\ell \le 50$ mm 50 mm $< \ell \le 150$ mm 150 mm $< \ell \le 300$ mm $\ell > 300$ mm For $t_2 \le 0{,}7\ t_1$, $\Delta\sigma_R$ may be increased by one class, but not over 56 (steel) or 20 (Al). If the plate or beam flange is subjected to in-plane bending, $\Delta\sigma_R$ has to be decreased by one class.	 56 50 45 40	 20 18 16 14

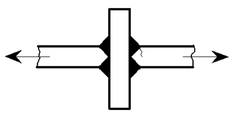
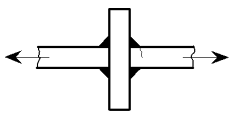
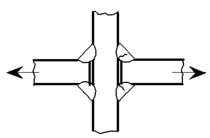
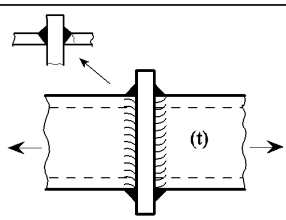
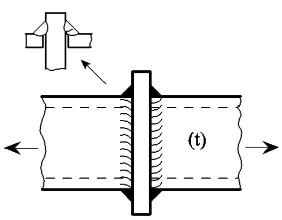
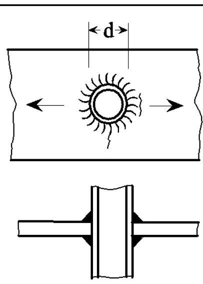
C. Non-load-carrying attachments				
Type No.	**Joint configuration showing mode of fatigue cracking and stress σ considered**	**Description of joint**	**FAT class $\Delta\sigma_R$** Steel	**FAT class $\Delta\sigma_R$** Al
C6		Longitudinal flat side gusset welded on plate edge or beam flange edge, with smooth transition (sniped end or radius); $c \le 2\, t_2$, max. 25 mm $r \ge 0{,}5\, h$ $r < 0{,}5\, h$ or $\varphi \le 20°$ $\varphi > 20°$ see joint type C5 For $t_2 \le 0{,}7\, t_1$, $\Delta\sigma_R$ may be increased by one class.	 50 45	 18 16
C6a		Longitudinal flat side gusset welded on plate edge or beam flange edge, with smooth transition radius $r/h > 1/3$ or $r \ge 150$ mm $1/6 < r/h < 1/3$ $r/h < 1/6$ Smooth transition radius formed by grinding the full penetration weld area in order to achieve a notch-free transition area. Final grinding shall be performed parallel to stress direction.	90 71 50	36 28 22
C7		Transverse stiffener with fillet welds (applicable for short and long stiffeners)	80	28
C8		Non-load-carrying shear connector	80	28
C9		End of long doubling plate on beam, welded ends (based on stress range in flange at weld toe) $t_D \le 0{,}8\, t$ $0{,}8\, t < t_D \le 1{,}5\, t$ $t_D > 1{,}5\, t$ The following features increase $\Delta\sigma_R$ by one class accordingly: – reinforced ends according to Section 15, Fig. 15.4 – weld toe angle $\le 30°$ – length of doubling ≤ 300 mm For length of doubling ≤ 150 mm, $\Delta\sigma_R$ may be increased by two classes.	56 50 45	20 18 16

Note:

Correction to type No. C9

reinforced ends according to App.C Figure 4.

---e-n-d---o-f---n-o-t-e---

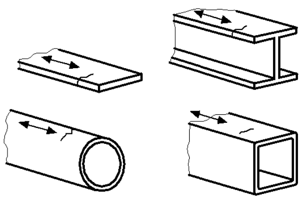
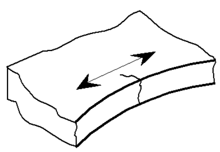
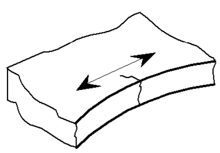
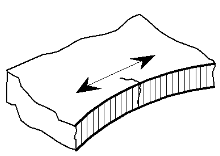
D. Cruciform joints and T-joints				
Type No.	Joint configuration showing mode of fatigue cracking and stress σ considered	Description of joint	FAT class $\Delta\sigma_R$	
			Steel	Al
D1		Cruciform or tee-joint K-butt welds with full penetration or with defined incomplete root penetration according to Section 15, Fig. 15.9. cruciform joint tee-joint	 71 80	 25 28
D2		Cruciform or tee-joint with transverse fillet welds, toe failure (root failure particularly for throat thickness $a < 0{,}7 \cdot t$, see joint type D3) cruciform joint tee-joint	 63 71	 22 25
D3		Welded metal in transverse load-carrying fillet welds at cruciform or tee-joint, root failure (based on stress range in weld throat), see also joint type No. D2 $a \geq t/3$ $a < t/3$ ***Note*** *Crack initiation at weld root*	 36 40	 12 14
D4		Full penetration weld at the connection between a hollow section (e.g. pillar) and a plate, for tubular section for rectangular hollow section For $t \leq 8$ mm, $\Delta\sigma_R$ has to be decreased by one class.	 56 50	 20 18
D5		Fillet weld at the connection between a hollow section (e.g. pillar) and a plate, for tubular section for rectangular hollow section The stress is to be related to the weld sectional area. For $t \leq 8$ mm, $\Delta\sigma_R$ has to be decreased by one class.	 45 40	 16 14
D6		Continuous butt or fillet weld connecting a pipe penetrating through a plate $d \leq 50$ mm $d > 50$ mm ***Note*** *For large diameters an assessment based on local stress is recommended.*	 71 63	 25 22

Note:

Correction to type No. D1

Cruciform or tee-joint K-butt welds with full penetration or with defined incomplete root penetration according to App.C Figure 9.

---e-n-d---o-f---n-o-t-e---

E. Unwelded base material				
Type No.	**Joint configuration showing mode of fatigue cracking and stress σ considered**	**Description of joint**	**FAT class $\Delta\sigma_R$**	
			Steel	**Al**
E1		Rolled or extruded plates and sections as well as seamless pipes, no surface or rolling defects	160 ($m_0 = 5$)	71 ($m_0 = 5$)
E2a		Plate edge sheared or machine-cut by any thermal process with surface free of cracks and notches, cutting edges chamfered or rounded by means of smooth grinding, groove direction parallel to the loading direction. Stress increase due to geometry of cut-outs to be considered by means of direct numerical calculation of the appertaining maximum notch stress range.	150 ($m_0 = 4$)	—
E2		Plate edge sheared or machine-cut by any thermal process with surface free of cracks and notches, cutting edges broken or rounded. Stress increase due to geometry of cut-outs to be considered. [1]	140 ($m_0 = 4$)	40 ($m_0 = 4$)
E3		Plate edge not meeting the requirements of type E2, but free from cracks and severe notches. Machine cut or sheared edge: Manually thermally cut: Stress increase due to geometry of cut-outs to be considered. [1]	125 ($m_0 = 3{,}5$) 100 ($m_0 = 3{,}5$)	36 ($m_0 = 3{,}5$) 32 ($m_0 = 3{,}5$)

[1] Stress concentrations caused by an opening to be considered as follows:

$\Delta\sigma_{max} = K_t \cdot \Delta\sigma_N$

K_t : Notch factor according to Section 3, J.

$\Delta\sigma_N$: Nominal stress range related to net section

alternatively direct determination of $\Delta\sigma_{max}$ from FE-calculation, especially in case of hatch openings or multiple arrangement of openings.

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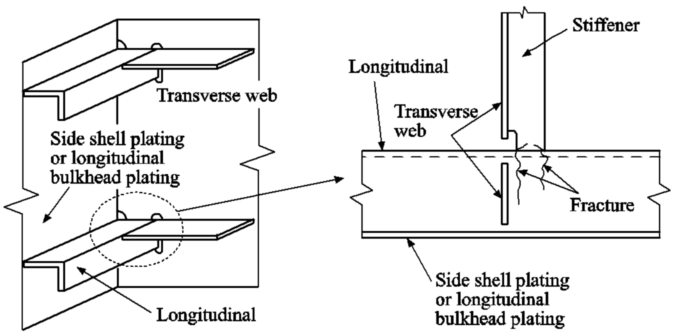
Note:

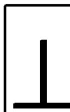
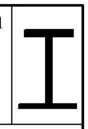
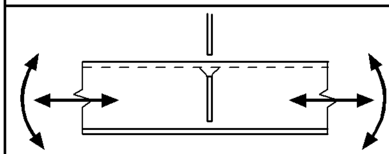
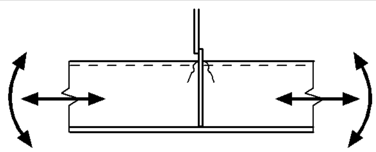
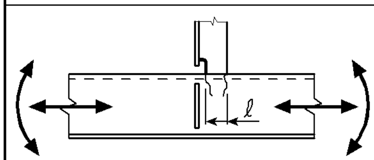
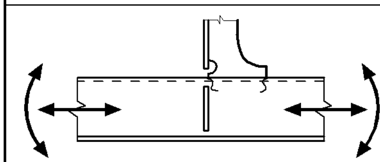
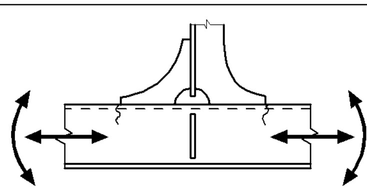
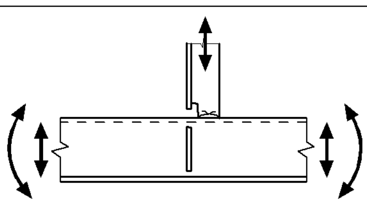
Correction to footnote 1

Notch factor according to App.A [8.5.3].

---e-n-d---o-f---n-o-t-e---

Table 7 Various intersections



Joint configuration Loads Locations being at risk for cracks	Description of joint		 [1]	 [1]	
		FAT class $\Delta\sigma_R$ steel			
	None watertight intersection without heel stiffener. For predominant longitudinal load only.	80	80	80	80
	Watertight intersection without heel stiffener (without cyclic load on the transverse member, see Section 9, B.4.1) For predominant longitudinal load only	71	71	71	71
 2	With heel stiffener direct connection $\ell \le 150$ $\ell > 150$ overlapping connection $\ell \le 150$ $\ell > 150$	 45 40 50 45	 56 50 50 45	 56 50 45 40	 63 56
 2	With heel stiffener and integrated bracket	45	56	56	63
	With heel stiffener and integrated bracket and with backing bracket direct connection overlapping connection	 50 56	 63 56	 63 50	 71
	With heel stiffener but considering the load transferred to the stiffener (see Section 9, B.4.9) crack initiation at weld toe crack initiation at weld root Stress increase due to eccentricity and shape of cut out has to be observed	80	71 40	71 40	71 40

1 Additional stresses due to asymmetric sections have to be observed, see Section 3,L.
2 To be increased by one class, when longitudinal loads only

Note:

Correction

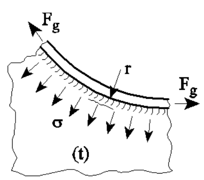
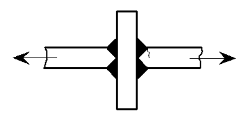
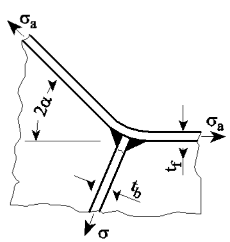
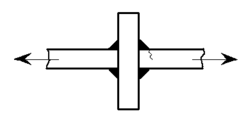
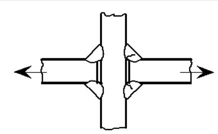
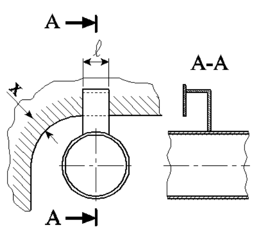
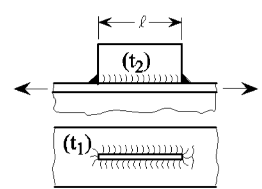
Watertight intersection without heel stiffener (without cyclic load on transverse member).

Correction to footnote 1

Additional stresses due to asymmetric sections have to be observed, see Sec.3 [6.2.2].

---e-n-d---o-f---n-o-t-e---

Table 8 Example of details

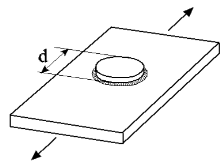
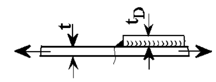
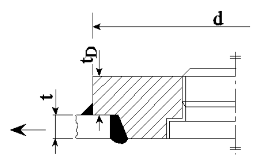
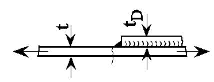
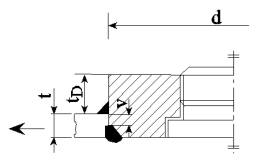
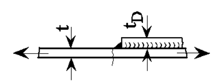
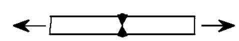
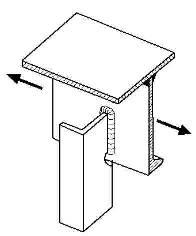
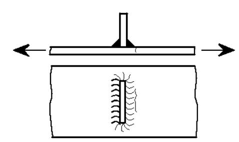
Structure or equipment detail	Description of structure or equipment detail	Type No.	Joint configuration showing mode of fatigue cracking and stress σ considered	Description of joint	FAT class $\Delta\sigma_R$ Steel
	Unstiffened flange to web joint, to be assessed according to type D1, D2 or D3, depending on the type of joint. The stress in the web is calculated using the force F_g in the flange as follows: $\sigma = \frac{F_g}{r \cdot t}$ Furthermore, the stress in longitudinal weld direction has to be assessed according to type B2 - B4. In case of additional shear or bending, also the highest principle stress may become relevant in the web, see 2.1.4.	D1		Cruciform or tee-joint K-butt welds with full penetration or with defined incomplete root penetration according to Sec. 19, Fig. 9. cruciform joint tee-joint	 71 80
	Joint at stiffened knuckle of a flange, to be assessed according to type D1, D2 or D3, depending on the type of joint. The stress in the stiffener at the knuckle can normally be calculated as follows: $\sigma = \sigma_a \frac{t_f}{t_b} 2 \sin \alpha$	D2		Cruciform or tee-joint with transverse fillet welds, toe failure (root failure particuarly for throat thickness a < 0.7 · t, see joint type D3) cruciform joint tee-joint	 63 71
		D3		Welded metal in transverse load-carrying fillet welds at cruciform or tee-joint, root failure (based on stress range in weld throat), see also joint type No. D2	36
	Holder welded in way of an opening and arranged parallel to the edge of the opening. not valid for hatch corner	C1		$\ell \leq 150$ mm In way of the rounded corner of an opening with the radius r a minimum distance x from the edge to be kept (hatched area): x [mm] = 15 + 0.175 · r [mm] 100 mm ≤ r ≤ 400 mm In case of an elliptical rounding the mean value ot both semiaxes to be applied	71

Note:

Correction to type No. D2

In case of additional shear or bending, also the highest principle stress may become relevant in the web, see App.C Figure 9.

---e-n-d---o-f---n-o-t-e---

Structure or equipment detail	Description of structure or equipment detail	Type No.	Joint configuration showing mode of fatigue cracking and stress σ considered	Description of joint	FAT class $\Delta\sigma_R$ steel
	Circular doubler plate with max. 150 mm diameter.	C9		$t_D \le 0.8\,t$ $0.8\,t < t_D \le 1.5\,t$ $t_D > 1.5\,t$	71 63 56
	Drain plugs with full penetration butt weld $d \le 150$ mm Assesment corresponding to doubling plate.	C9		$t_D \le 0.8\,t$ $0.8\,t < t_D \le 1.5\,t$ $t_D > 1.5\,t$ For $d > 150$ mm $\Delta\sigma_R$ has to be decreased by one class	71 63 56
	Drain plugs with partial penetration butt weld and a defined root gap $d \le 150$ mm For $v < 0.4\,t$ or $v < 0.4\,t_D$	C9		$0.2\,t < t_D \le 0.8\,t$ $0.8\,t < t_D \le 1.5\,t$ $1.5\,t < t_D < 2.0\,t$ For $d > 150$ mm $\Delta\sigma_R$ has to be decreased by one class	50 45 40
	For $v \ge 0.4\,t$ and $v \ge 0.4\,t_D$	A7		Partial penetration butt weld; the stress is to be related to the weld throat sectional area, weld overfill not to be taken into account	36
	The detail category is also valid for not fully circumferential welded holders For stiffeners loaded in bending $\Delta\sigma_R$ to be downgraded by one class	C7		Transverse stiffener with fillet welds (applicable for short and long stiffeners)	80

APPENDIX E NON-LINEAR STRENGTH ANALYSIS

1 Progressive collapse analysis

A progressive collapse analysis shall be used to calculate the ultimate vertical bending moments of a ship's effective transverse section. The procedure shall be based on a simplified incremental-iterative approach where the capacities are defined as the peaks of the resulting moment-curvature curve (M-χ) in hogging (positive) and sagging (negative) conditions, i.e. χ is the hull girder curvature [1/m]. See Figure 1.

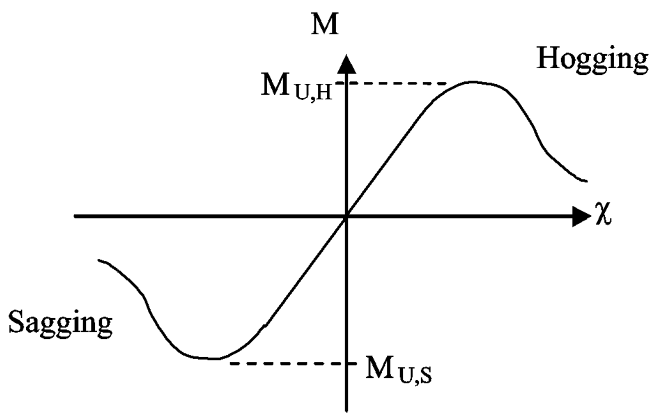


Figure 1 Moment-curvature curve

The main steps to be used in the incremental-iterative approach are summarized as follows:

Step 1 The ship's transverse section shall be divided into plate-stiffener combinations (see [1.3] (a)) and hard corners (see [1.3] (b)).

Step 2 The average stress – average strain relationships σ_{CRk}-ε for all structural elements (i.e. stiffener-plate combinations and hard corners) shall be defined, where the subscript *k* refers to the modes 0, 1, 2, 3 or 4, as applicable (see [1.1]).

Step 3 The initial and incremental value of curvature Δχ shall be defined by the following equation:

$$\Delta_{\chi} = \frac{0.05 \cdot \frac{R_y}{E}}{z_D - z_{NA,e}}$$

where:

R_{eH} = minimum nominal yield point of structural elements in the strength deck [MPa]
z_D = *z* co-ordinate of strength deck at side [m]
$z_{NA,e}$ = *z* co-ordinate of elastic neutral axis for the ship's transverse section [m].

Step 4 For the value of curvature, $\chi_j = \chi_{j-1} + \Delta\chi$, the average strain $\varepsilon_{Ei,j} = \chi_j z_i$ and corresponding average stress $\sigma_{i,j}$ shall be defined for each structural element i (see [1.1]). For structural elements under tension, $\sigma_{i,j} = \sigma_{CR0}$ (see [1.2]). For plate-stiffener combinations under compression, $\sigma_{i,j}$ = minimum [σ_{CR1}, σ_{CR2}, σ_{CR3}] (see [1.3] (a)). For hard corners under compression, $\sigma_{i,j} = \sigma_{CR4}$ (see [1.3] (b)).

z_i = z co-ordinate of i^{th} structural element [m] relative to basis, see also Figure 3.

Step 5 For the value of curvature, $\chi_j = \chi_{j-1} + \Delta\chi$, the height of the neutral axis $z_{NA,j}$ shall be determined iteratively through force equilibrium over the ship's transverse section:

$$\sum_{i=1}^{m} A_i \sigma_{i,j} = \sum_{i=1}^{n} A_i \sigma_{i,j}$$

where:

m = the number of structural elements located above $z_{NA,j}$
n = the number of structural elements located below $z_{NA,j}$
A_i = cross-sectional area of i^{th} plate-stiffener combination or hard corner.

Step 6 For the value of curvature, $\chi_j = \chi_{j-1} + \Delta\chi$, the corresponding bending moment shall be calculated by summing the contributions of all structural elements within the ship's transverse section:

$$M_{U,j} = \sum \sigma_{i,j} A_i \left(z_{NA,j} - z_i\right)$$

Steps 4 through 6 shall be repeated for increasing increments of curvature until the peaks in the M-χ curve are well defined. The ultimate vertical bending moments $M_{U,H}$ and $M_{U,S}$ shall be taken as the peak values of the M-χ curve.

1.1 Average stress - average strain curves

A typical average stress – average strain curve σ_{CRk}-ε for a structural element within a ship's transverse section is shown in Figure 2, where the subscript k refers to the modes 0, 1, 2, 3 or 4, as applicable.

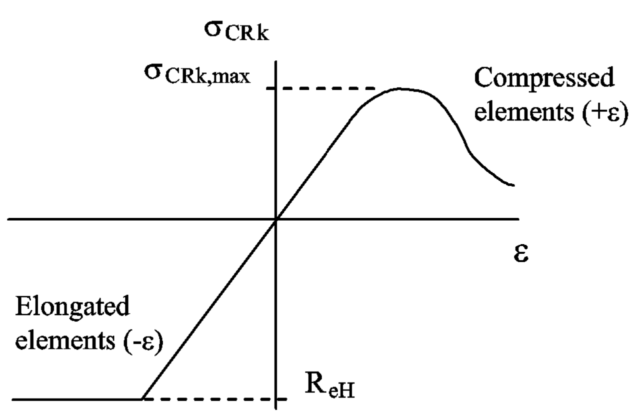


Figure 2 Typical average stress - average strain curve

1.2 Negative strain (σ_{CR0}-ε)

The portion of the curve corresponding to negative strain (i.e. tension) is in every case to be based on elasto-plastic behaviour (i.e. material yielding) according to the following:

$$\sigma_{CR0} = \Phi R_{eH} \text{ [MPa]}$$

where:

Φ = edge function

= −1 for ε < −1

= ε for −1 ≤ ε ≤ 0

ε = relative strain

= $\frac{\varepsilon_E}{\varepsilon_Y}$

ε_E = element strain

ε_Y = strain at yield stress in the element

= $\frac{R_y}{E}$

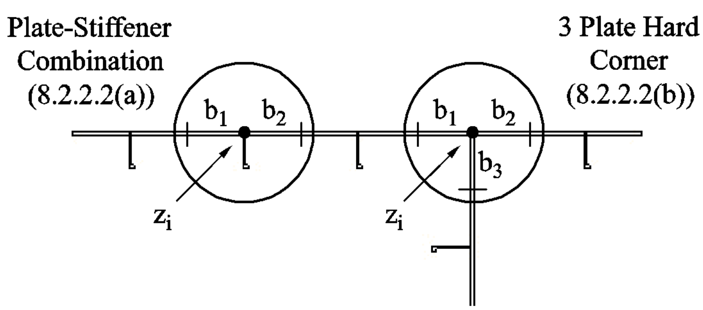


Figure 3 Structural elements

1.3 Positive strain

The portion of the curve corresponding to positive strain (i.e. compression) shall be based on some mode of collapse behaviour (i.e. buckling) for two types of structural elements; (a) plate-stiffener combinations and (b) hard corners see Figure 3.

(a) Plate-stiffener combinations (σ_{CR1}-ε, σ_{CR2}-ε, σ_{CR3}-ε)

Plate-stiffener combinations are comprised of a single stiffener together with the attached plating from adjacent plate fields. Under positive strain, three average stress – average strain curves shall be defined for each plate-stiffener combination based on beam column buckling (σ_{CR1}-ε), torsional buckling (σ_{CR2}-ε) and web/flange local buckling (σ_{CR3}-ε).

(i) Beam column buckling σ_{CR1}-ε

The positive strain portion of the average stress – average strain curve σ_{CR1}-ε based on beam column buckling of plate-stiffener combinations is described according to the following:

$$\sigma_{CR1} = \Phi \cdot R_y \cdot \kappa_{BC} \cdot \frac{A_{Stif} + b_{m,1} \cdot \frac{t_1}{2} + b_{m,2} \cdot \frac{t_2}{2}}{A_{Stif} + b_1 \cdot \frac{t_1}{2} + b_2 \cdot \frac{t_1}{2}}$$

where:

Φ	=	edge function
	=	ε for 0 ≤ ε ≤ 1.0
	=	1.0 for ε > 1
κ_{BC}	=	reduction factor
	=	1.0 for $\lambda_K \le \lambda_0$
	=	$\frac{1}{k_D + \sqrt{k_D^2 - \lambda_K^2}}$ for $\lambda_K > \lambda_0$
k_D	=	$0.5\,(1 + \eta_p\,(\lambda_K - \lambda_0) + \lambda_K^2)$
λ_K	=	$\sqrt{\frac{\varepsilon_E a^2 A_x}{\pi^2 I_x} \cdot 10^{-4}}$
a	=	length of stiffener [mm]
A_x	=	sectional area of stiffener with attached shell plating of breadth 0.5 ($b_{m,1}$ + $b_{m,2}$) [mm^2]
I_x	=	moment of inertia of stiffener with attached shell plating of breadth ($b_{m,1}/2$ + $b_{m,2}/2$) [cm^4]
$b_{m,1}$, $b_{m,2}$	=	effective breadths of single plate fields on sides 1 and 2 of stiffener [mm] according to App.B [2.2], in general based on Load Case 1 of App.B Table 3, where the reference degree of slenderness shall be defined as $\lambda = \sqrt{\frac{\varepsilon_E}{0.9\left(\frac{t}{b}\right)^2 K}}$
b_1, b_2	=	breadths of single plate fields on sides 1 and 2 of stiffener [mm], see also Figure 3
t_1, t_2	=	thicknesses of single plate fields on sides 1 and 2 of stiffener [mm]
A_{Stif}	=	sectional area of the stiffener without attached plating [mm^2]

for steel:

λ_0 = 0.20

n_P = 0.21

for aluminium without heat treatment (i.e. 5000 series):

λ_0 = 0.00

n_p = 0.32

for aluminium with heat treatment (i.e. 6000 series):

λ_0 = 0.10
n_p = 0.20

(ii) Torsional buckling σ_{CR2}-ε

The positive strain portion of the average stress – average strain curve σ_{CR2}-ε based on torsional buckling of plate-stiffener combinations is described according to the following:

$$\sigma_{CR2} = \Phi \cdot R_y \cdot \frac{A_{Stif} \cdot \kappa_T + b_{m,1} \cdot \frac{t_1}{2} + b_{m,2} \cdot \frac{t_2}{2}}{A_{Stif} + b_1 \cdot \frac{t_1}{2} + b_2 \cdot \frac{t_1}{2}}$$

where:

κ_T = reduction factor according to App.B [3.3.1].

(iii) Web/flange local buckling σ_{CR3}-ε

The positive strain portion of the average stress – average strain curve σ_{CR3}-ε based on web/flange local buckling of plate-stiffener combinations is described according to the following:

$$\sigma_{CR3} = \Phi \cdot R_y \cdot \frac{h_{w,m} \cdot t_w + b_{f,m} \cdot t_f + b_{m,1} \cdot \frac{t_1}{2} + b_{m,2} \cdot \frac{t_2}{2}}{h_w \cdot t_w + b_{f,m} \cdot t_f + b_1 \cdot \frac{t_1}{2} + b_2 \cdot \frac{t_1}{2}}$$

where:

$h_{w,m}$, $b_{f,m}$ = effective width of web/flange plating [mm] according to App.B [2.2] (generally based on Load Case 3 of App.B Table 3 for flat bars and flanges, otherwise Load Case 1) where the reference degree of slenderness shall be defined as

$$\lambda = \sqrt{\frac{\varepsilon_E}{0.9\left(\frac{t}{b}\right)^2 K}}$$

h_w = web height [mm]
t_w = web thickness [mm]
b_f = flange breadth, where applicable [mm]
t_f = flange thickness, where applicable [mm].

(b) Hard corners (σ_{CR4}-ε)

Hard corners are sturdy structural elements comprised of plates not lying in the same plane. Bilge strakes (i.e. one curved plate), sheer strake-deck stringer connections (i.e. two plane plates) and bulkhead-deck connections (i.e. three plane plates) are typical hard corners. Under positive strain, single average stress – average strain curves shall be defined for hard corners based on plate buckling (σ_{CR4}-ε).

(i) Plate buckling σ_{CR4}-ε

$$\sigma_{CR4} = \Phi R_y \frac{\sum_{i=1}^{n} b_{m,i} \cdot t_i}{\sum_{i=1}^{n} b_i \cdot t_i}$$

where:

$b_{m,i}$ = effective breadths of single plate fields [mm] according to App.B [2.2], as applicable, in general based on applicable Load Cases in App.B Table 3 and App.B Table 4, where the reference degree of slenderness shall be defined as

$$\lambda = \sqrt{\frac{\varepsilon_E}{0.9\left(\frac{t}{b}\right)^2 K}}$$

b_i = breadth of single plate fields [mm], see also Figure 3

t_i = thickness of single plate fields [mm]

n = number of plates comprising hard corner.

CHANGES – HISTORIC

December 2015 edition

This is a new document.

The rules enter into force 1 July 2016.

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