# SECTION 4 STIFFENERS

## Symbols

E : Young's modulus, in N/mm², to be taken equal to:

- for steels in general:
  $E = 2{,}06.10^5$ N/mm²
- for stainless steels:
  $E = 1{,}95.10^5$ N/mm²
- for aluminium alloys:
  $E = 7{,}0.10^4$ N/mm²

## 1 General

### 1.1 Geometric properties

#### 1.1.1 Built section

The geometric properties of built sections as shown in Fig 1 may be calculated as indicated in the following formulae. These formulae are applicable provided that:

$A_a \geq t_f b_f$

$\frac{h_w}{t_p} \geq 10$

$\frac{h_w}{t_f} \geq 10$

where:

$A_a$ : Sectional area, in mm², of the attached plating.

The section modulus of a built section with attached plating is to be obtained, in cm³, from the following formula:

$$w = \frac{h_w t_f b_f}{1000} + \frac{t_W h_W^2}{6000}\left(1 + \frac{A_a - t_f b_f}{A_a + \frac{t_W h_W}{2}}\right)$$

The distance from face plate to neutral axis is to be obtained, in cm, from the following formula:

$$v = \frac{h_W(A_a + 0,5 t_W h_W)}{10(A_a + t_f b_f + t_W h_W)}$$

The moment of inertia of a built section with attached plating is to be obtained, in cm⁴, from the following formula:

$I = w\,v$

The shear sectional area of a built section with attached plating is to be obtained, in cm², from the following formula:

$$A_{Sh} = \frac{h_w t_w}{100}$$

#### 1.1.2 Bulb section: equivalent angle profile

A bulb section may be taken as equivalent to an angle profile.

The dimensions of the equivalent angle profile are to be obtained, in mm, from the following formulae:

$$h_w = h'_w - \frac{h'_w}{9,2} + 2$$

$$t_w = t'_w$$

$$b_f = \alpha\left[t'_w + \frac{h'_w}{6,7} - 2\right]$$

$$t_f = \frac{h'_w}{9,2} - 2$$

where:

$h'_w$, $t'_w$ : Height and thickness of the bulb section, in mm, as shown in Fig 2

$\alpha$ : Coefficient equal to:

$1,1 + \frac{(120 - h'_w)^2}{3000}$ for $h'_w \leq 120$

$1$ for $h'_w > 120$

**Figure 1 : Dimensions of a built section**

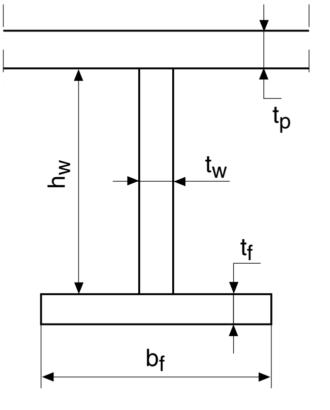
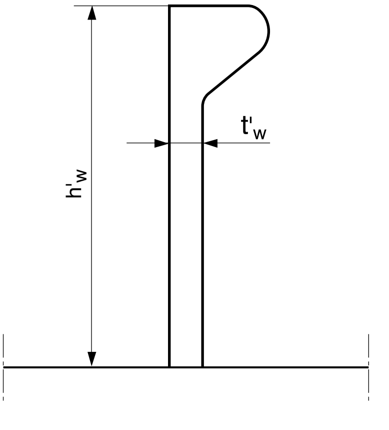



**Figure 2 : Dimensions of a bulb section**



### 1.1.3 Stiffener not perpendicular to the attached plating

Where the stiffener is not perpendicular to the attached plating, the actual net section modulus may be obtained, in cm³, from the following formula:

$$w = w_0 \sin\alpha$$

where:

$w_0$ : Actual net section modulus, in cm³, of the stiffener assumed to be perpendicular to the plating

$\alpha$ : Angle between the stiffener web and the attached plating.

## 1.2 Ordinary stiffeners

### 1.2.1 Span

The span $\ell$, in m, of ordinary stiffeners is to be taken as indicated in Ch 4, Sec 4, [3].

### 1.2.2 Attached plating for lateral loading

The width of the attached plating to be considered for the yielding check of ordinary stiffeners is to be obtained, in m, from the following formulae, where s is the spacing between ordinary stiffeners, in m:

- where the plating extends on both sides of the ordinary stiffener:

  $b_P = s$

- where the plating extends on one side of the ordinary stiffener (i.e. ordinary stiffeners bounding openings):

  $b_P = 0,5s$

### 1.2.3 Attached plating for buckling check

Where ordinary stiffeners are sustaining compression stress, the width of attached plate and the buckling check method are given in [2].

**1.2.4** Where ordinary stiffeners are continuous through primary supporting members, their connection to the web of the primary supporting member is to be in accordance with [1.6].

**1.2.5** As a rule, where ordinary stiffeners are cut at primary supporting members, brackets are to be fitted to ensure the structural continuity. Their net section modulus and their net sectional area are to be not less than those of the ordinary stiffeners.

## 1.3 Primary supporting members

### 1.3.1 Span

The span $\ell$, in m, of primary supporting members is to be taken as indicated in Ch 4, Sec 4, [3].

### 1.3.2 Attached plating

The width of the attached plating to be considered for the yielding check of primary supporting members analysed through beam structural models is to be obtained, in m, from the following formulae, where s is the spacing of the primary supporting members:

- where the plating extends on both sides of the primary supporting member:

  $b_P = \min (s; 0,2\ell)$

- where the plating extends on one side of the primary supporting member (i.e. primary supporting members bounding openings):

  $b_P = 0,5 \min (s; 0,2\ell)$

**1.3.3** The web shear area of primary supporting members is to take into account the section reduction due to cut-outs provide for ordinary stiffeners, if relevant.

**1.3.4** Cut-outs for the passage of ordinary stiffeners are to be as small as possible and well rounded with smooth edges.

In general, the depth of cut-outs is to be not greater than 50% of the depth of the primary supporting member.

**1.3.5** Where openings such as lightening holes or duct routing for pipes, electrical cable... are cut in primary supporting members, they are to be equidistant from the face plate and the attached plate. As a rule, their height is not to be more than 20% of the primary supporting member web height.

**1.3.6** Openings may not be fitted in way of toes of end brackets.

**1.3.7** Over half of the span in the middle of the primary supporting members, the length of openings is to be not greater than the distance between adjacent openings.

At the ends of the span, the length of openings is to be not greater than 25% of the distance between adjacent openings.

## 1.4 Large openings in primary supporting members

**1.4.1** In the case of large openings as shown in Fig 3, the secondary stresses in primary supporting members are to be considered for the reinforcement of the openings.

The secondary stresses may be calculated in accordance with the following procedure.

Members (1) and (2) are subjected to the following forces, moments and stresses:

$$F = \frac{M_A + M_B}{2d}$$

$$m_1 = \left|\frac{M_A - M_B}{2}\right| K_1$$

$$m_2 = \left|\frac{M_A - M_B}{2}\right| K_2$$

$$\sigma_{F1} = 10\frac{F}{S_1}$$

$$\sigma_{F2} = 10\frac{F}{S_2}$$

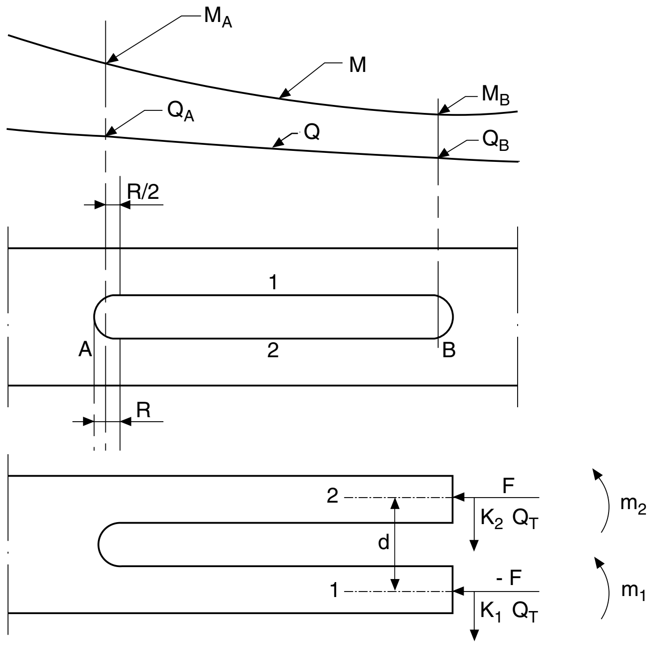
$$\sigma_{m1} = \frac{m_1}{w_1}10^3$$

$$\sigma_{m2} = \frac{m_2}{w_2}10^3$$

$$\tau_1 = 10\frac{K_1 Q_T}{S_{w1}}$$

$$\tau_2 = 10\frac{K_2 Q_T}{S_{w2}}$$

**Figure 3 : Large openings in primary supporting members - Secondary stresses**




where:

$M_A$, $M_B$ : Bending moments, in kN.m, in sections A and B of the primary supporting member

$m_1$, $m_2$ : Bending moments, in kN.m, in (1) and (2)

d : Distance, in m, between the neutral axes of (1) and (2)

$\sigma_{F1}$, $\sigma_{F2}$ : Axial stresses, in N/mm², in (1) and (2)

$\sigma_{m1}$, $\sigma_{m2}$ : Bending stresses, in N/mm², in (1) and (2)

$Q_T$ : Shear force, in kN, equal to $Q_A$ or $Q_B$, whichever is greater

$\tau_1$, $\tau_2$ : Shear stresses, in N/mm², in (1) and (2)

$w_1$, $w_2$ : Net section moduli, in cm³, of (1) and (2)

$S_1$, $S_2$ : Net sectional areas, in cm², of (1) and (2)

$S_{w1}$, $S_{w2}$ : Net sectional areas, in cm², of webs in (1) and (2)

$I_1$, $I_2$ : Net moments of inertia, in cm⁴, of (1) and (2) with attached plating

$$K_1 = \frac{I_1}{I_1 + I_2}$$

$$K_2 = \frac{I_2}{I_1 + I_2}$$

The combined stress $\sigma_C$ calculated at the ends of members (1) and (2) is to be obtained from the following formula:

$$\sigma_c = \sqrt{(\sigma_F + \sigma_m)^2 + 3\tau^2}$$

The combined stress $\sigma_C$ is to comply with the checking criteria in Ch 4, Sec 3, Tab 2 or Ch 4, Sec 3, Tab 4 applicable. Where these checking criteria are not complied with, the cut-out is to be reinforced according to one of the solutions shown in Fig 4 to Fig 6:

- continuous face plate (solution 1): see Fig 4
- straight face plate (solution 2): see Fig 5
- compensation of the opening (solution 3): see Fig 6
- combination of the above solutions.

Other arrangements may be accepted provided they are supported by direct calculations submitted to the Society for review.

## 1.5 Web stiffening arrangement for primary supporting members

**1.5.1** Webs of primary supporting members are generally to be stiffened where the height, in mm, is greater than 100 t, where t is the web thickness, in mm, of the primary supporting member.

In general, the web stiffeners of primary supporting members are to be spaced not more than 110 t.

**1.5.2** The section modulus of web stiffeners of non-watertight primary supporting members is to be not less than the value obtained, in cm³, from the following formula:

$$w = 2{,}5 s^2 t S_s^2$$

where:

s : Length, in m, of web stiffeners

t : Web thickness, in mm, of the primary supporting member

$S_s$ : Spacing, in m, of web stiffeners.

Moreover, web stiffeners located in areas subject to compression stresses are to be checked for buckling in accordance with [2].

**Figure 4 : Stiffening of large openings in primary supporting members - Solution 1**

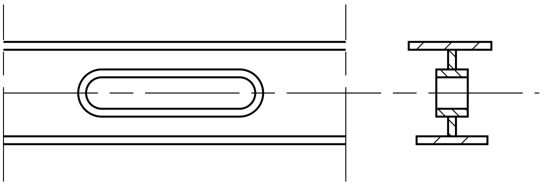


**Figure 5 : Stiffening of large openings in primary supporting members - Solution 2**

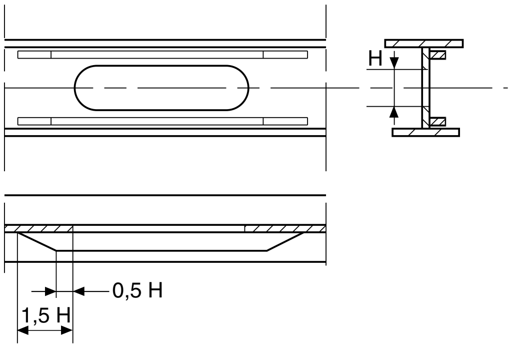
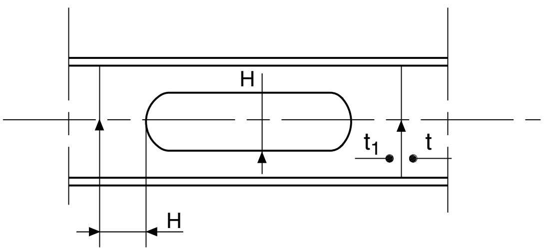



**Figure 6 : Stiffening of large openings in primary supporting members - Solution 3**

Inserted plate




**1.5.3** Tripping brackets (see Fig 7) welded to the face plate are generally to be fitted:

- every fourth spacing of ordinary stiffeners
- at the toe of end brackets
- at rounded face plates
- in way of cross ties
- in way of concentrated loads.

Where the width of the symmetrical face plate is greater than 400 mm, backing brackets are to be fitted in way of the tripping brackets.

**1.5.4** In general, the width of the primary supporting member face plate is to be not less than one tenth of the depth of the web, where tripping brackets are spaced as specified in [1.5.3].

**1.5.5** The arm length of tripping brackets is to be not less than the greater of the following values, in m:

$$d = 0{,}38b$$

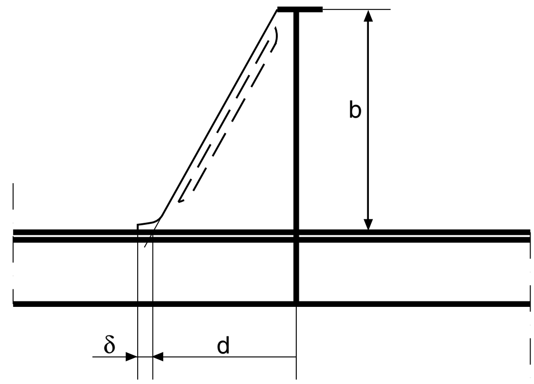
$$d = 0{,}85b\sqrt{\frac{s_t}{t}}$$

where:

| | | |
|---|---|---|
| b | : | Height, in m, of tripping brackets, shown in Fig 7 |
| $s_t$ | : | Spacing, in m, of tripping brackets |
| t | : | Thickness, in mm, of tripping brackets. |

It is recommended that the bracket toe should be designed as shown in Fig 7.

**Figure 7 : Primary supporting member: web stiffener in way of ordinary stiffener**




**1.5.6** Steel tripping brackets with a thickness, in mm, less than $16{,}5L_b$ are to be flanged or stiffened by a welded face plate.

Aluminium tripping brackets with a thickness, in mm, less than $22L_b$ are to be flanged or stiffened by a welded face plate.

The sectional area, in cm², of the flanged edge or the face plate is to be not less than $10L_b$, where $L_b$ is the length, in m, of the free edge of the bracket.

Where the depth of tripping brackets is greater than 3 m, an additional stiffener is to be fitted parallel to the bracket free edge.

## 1.6 Connections of ordinary stiffeners and primary supporting members

**1.6.1** Where ordinary stiffeners are continuous through primary supporting members, they are to be connected to the web plating so as to ensure proper transmission of loads, e.g. by means of one of the connection details shown in Fig 8 to Fig 11.

Connection details other than those shown in Fig 8 to Fig 11 may be considered by the Society on a case by case basis. In some cases, the Society may require the details to be supported by direct calculations submitted for review.

**Figure 8 : End connection of ordinary stiffener Without collar plate**

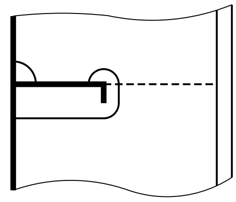


**Figure 9 : End connection of ordinary stiffener Collar plate**

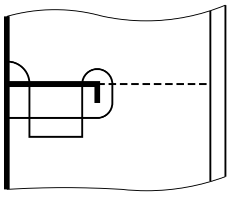


**Figure 10 : End connection of ordinary stiffener One large collar plate**

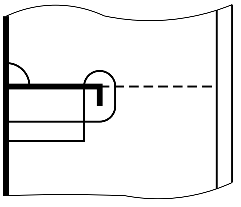


**Figure 11 : End connection of ordinary stiffener Two large collar plates**

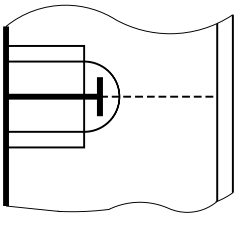
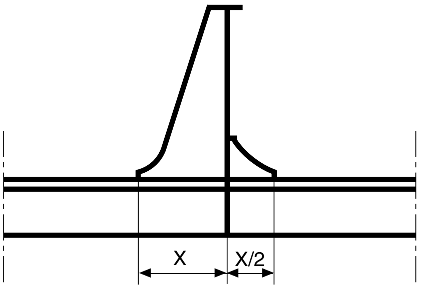


**Figure 12 : End connection of ordinary stiffener Backing bracket**




**1.6.2** Where ordinary stiffeners are cut at primary supporting members, brackets are to be fitted to ensure the structural continuity. Their section modulus and their sectional area are to be not less than those of the ordinary stiffeners.

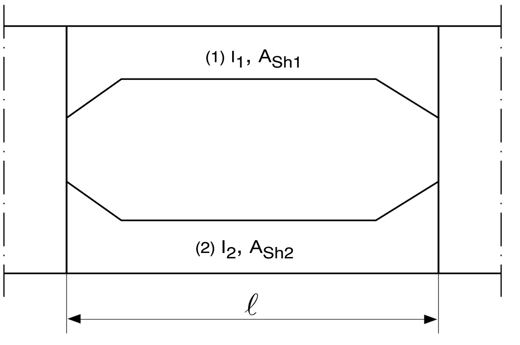
The thickness of brackets is to be not less than that of ordinary stiffeners.

Steel brackets with thickness, in mm, less than $16,5L_b$, where $L_b$ is the length, in m, of the free edge of the end bracket, are to be flanged or stiffened by a welded face plate. The sectional area, in $cm^2$, of the flanged edge or face plate is to be at least equal to $10L_b$ .

Aluminium brackets with thickness, in mm, less than $22L_b$, where $L_b$ is the length, in m, of the free edge of the end bracket, are to be flanged or stiffened by a welded face plate. The sectional area, in $cm^2$, of the flanged edge or face plate is to be at least equal to $10L_b$ .

**1.6.3** Where necessary (high stress level), the Society may require backing brackets to be fitted, as shown in Fig 12, in order to improve the fatigue strength of the connection.

**Figure 13 : Large openings in the web of primary supporting members**




**1.6.4** Where large openings are fitted in the web of primary supporting members (see Fig 13), their influence is to be taken into account by assigning an equivalent shear sectional area to the primary supporting member.

This equivalent shear sectional area is to be obtained, in $cm^2$, from the following formula:

$$A_{Sh} = \frac{A_{Sh1}}{1 + \dfrac{0{,}0032\,\ell^2 A_{Sh1}}{I_1}} + \frac{A_{Sh2}}{1 + \dfrac{0{,}0032\,\ell^2 A_{Sh2}}{I_2}}$$

where (see Fig 13):

- $I_1$, $I_2$ : Moments of inertia, in $cm^4$, of deep webs (1) and (2), respectively, with attached plating around their neutral axes parallel to the plating
- $A_{Sh1}$, $A_{Sh2}$ : Shear sectional areas, in $cm^2$, of deep webs (1) and (2), respectively, to be calculated according to [1.1.1].
- $\ell$ : Span, in cm, of deep webs (1) and (2).

## 1.7 Bracketed end connections

**1.7.1** Arm lengths of end brackets are to be equal, as far as practicable.

With the exception of primary supporting members of transversely framed single sides, the height of end brackets is to be not less than that of the primary supporting member.

**1.7.2** The thickness of the end bracket web is generally to be not less than that of the primary supporting member web.

**1.7.3** The scantlings of end brackets are generally to be such that the section modulus of the primary supporting member with end brackets is not less than that of the primary supporting member at mid-span.

**1.7.4** The width, in mm, of the face plate of end brackets is to be not less than $50(L_b+1)$, where $L_b$ is the length, in m, of the free edge of the end bracket.

Moreover, the thickness of the face plate is to be not less than that of the bracket web.

**1.7.5** Where necessary, face plate of end brackets are to be symetrical.

In such case, the following prescriptions are to be complied with, as a rule:

- face plates are to be snipped at the ends with total angle not greater than 30°
- the width of face plate at ends is not to exceed 25 mm
- face plate with 20 mm thickness or more are to be tapered at ends on half the thickness
- radius of curved face plate is to be as large as possible
- collar plate is fitted in way of bracket toes
- fillet weld throat thickness is to be not less than t/2, where t is the tickness at the bracket toe.

### 1.8 Recommended dimensions of steel ordinary stiffeners

#### 1.8.1 Flat bar

The dimensions of a flat bar ordinary stiffener (see Fig 14) are to comply with the following requirement:

$$\frac{h_w}{t_w} \le 20\sqrt{k}$$

#### 1.8.2 T-section

The dimensions of a T-section ordinary stiffener (see Fig 15) are to comply with the following two requirements:

$$\frac{h_w}{t_w} \le 55\sqrt{k}$$

$$\frac{b_f}{t_f} \le 33\sqrt{k}$$

$$b_f t_f \ge \frac{h_w t_w}{6}$$

#### 1.8.3 Angle

The dimensions of a steel angle ordinary stiffener (see Fig 16) are to comply with the following two requirements:

$$\frac{h_w}{t_w} \le 55\sqrt{k}$$

$$\frac{b_f}{t_f} \le 16{,}5\sqrt{k}$$

$$b_f t_f \ge \frac{h_w t_w}{6}$$

### 1.9 Recommended dimensions of aluminium ordinary stiffeners

#### 1.9.1 Flat bar

The dimensions of a flat bar ordinary stiffener (see Fig 14) are to comply with the following requirement:

$$\frac{h_w}{t_w} \le 15\sqrt{k}$$

#### 1.9.2 T-section

The dimensions of a T-section ordinary stiffener (see Fig 15) are to comply with the following two requirements:

$$\frac{h_w}{t_w} \le 33\sqrt{k}$$

$$\frac{b_f}{t_f} \le 21\sqrt{k}$$

$$b_f t_f \ge \frac{h_w t_w}{6}$$

## 2 Buckling check

### 2.1 Width of attached plating

**2.1.1** The width of the attached plating to be considered for the buckling check of ordinary stiffeners is to be obtained, in m, from the following formulae:

- where no local buckling occurs on the attached plating:

  $b_e = s$
- where local buckling occurs on the attached plating:

  $$b_e = \left(\frac{2{,}25}{\beta_e} - \frac{1{,}25}{\beta_e^2}\right)s$$

  to be taken not greater than s

where:

$$\beta_e = \frac{s}{t_p}\sqrt{\frac{\sigma_b}{E}}10^3$$

$\sigma_b$ : Global hull girder compression stress $\sigma_X$ or $\sigma_Y$, in N/mm², acting on the plate panel, defined in Ch 4, Sec 4, according to the direction x or y considered.

**Figure 14 : Dimensions of a flat bar**

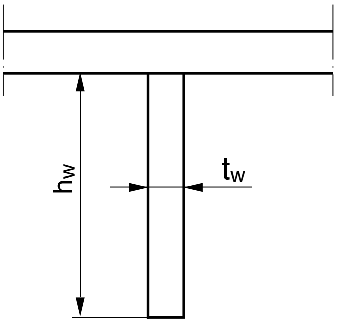



**Figure 15 : Dimensions of a T-section**

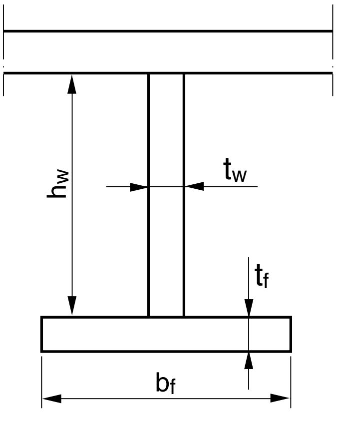
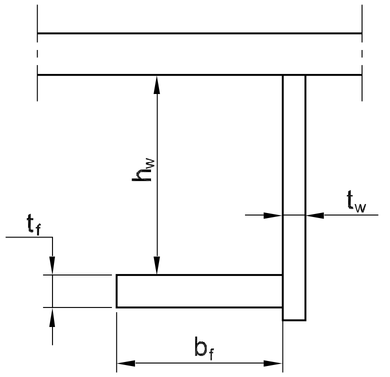


**Figure 16 : Dimensions of an angle**




## 2.2 Critical stress for steel members

### 2.2.1 General

The critical buckling stress is to be obtained, in N/mm², from the following formulae:

$$\sigma_c = \sigma_E \quad \text{for } \sigma_E \le \frac{R_{p0,2}}{2}$$

$$\sigma_c = R_{p0,2}\left(1 - \frac{R_{p0,2}}{4\sigma_E}\right) \quad \text{for } \sigma_E > \frac{R_{p0,2}}{2}$$

where:

$\sigma_E = \min(\sigma_{E1}, \sigma_{E2}, \sigma_{E3})$

$\sigma_{E1}$ : Euler column buckling stress, in N/mm², given in [2.2.2]

$\sigma_{E2}$ : Euler torsional buckling stress, in N/mm², given in [2.2.3]

$\sigma_{E3}$ : Euler web buckling stress, in N/mm², given in [2.2.4]

$R_{p0,2}$ : Minimum guaranteed yield stress, in N/mm², of the stiffener material, defined in Ch 4, Sec 3.

### 2.2.2 Column buckling of axially loaded stiffeners

The Euler column buckling stress is obtained, in N/mm², from the following formula:

$$\sigma_E = \pi^2 E \frac{I_e}{A_e \ell^2} 10^{-4}$$

$I_e$ : Moment of inertia, in cm⁴, of the stiffener with attached shell plating of width $b_e$, about its neutral axis parallel to the plating

$A_e$ : Sectional area, in cm², of the stiffener with attached plating of width $b_e$.

### 2.2.3 Torsional buckling of axially loaded stiffeners

The Euler torsional buckling stresses is obtained, in N/mm², from the following formula:

$$\sigma_E = \frac{\pi^2 E I_w}{10^4 I_p \ell^2}\left(\frac{K_C}{m^2} + m^2\right) + 0{,}385 E \frac{I_t}{I_p}$$

where:

$I_w$ : Sectorial moment of inertia, in cm⁶, of the stiffener about its connection to the attached plating:

- for flat bars:

$$I_w = \frac{h_w^3 t_w^3}{36} 10^{-6}$$

- for T-sections:

$$I_w = \frac{t_f b_f^3 h_w^2}{12} 10^{-6}$$

- for angles and bulb sections:

$$I_w = \frac{b_f^3 h_w^2}{12(b_f + h_w)^2}[t_f b_f^2 + 2 b_f h_w + 4 h_w^2 + 3 t_w b_f h_w]\, 10^{-6}$$

$I_p$ : Polar moment of inertia, in cm⁴, of the stiffener about its connection to the attached plating:

- for flat bars:

$$I_p = \frac{h_w^3 t_w}{3} 10^{-4}$$

- for stiffeners with face plate:

$$I_p = \left(\frac{h_w^3 t_w}{3} + h_w^2 b_f t_f\right) 10^{-4}$$

$I_t$ : St. Venant's moment of inertia, in cm⁴, of the stiffener without attached plating:

- for flat bars:

$$I_t = \frac{h_w t_w^3}{3} 10^{-4}$$

- for stiffeners with face plate:

$$I_t = \frac{1}{3}\left[h_w t_w^3 + b_f t_f^3\left(1 - 0{,}63\frac{t_f}{b_f}\right)\right] 10^{-4}$$

m : Number of half waves, to be taken equal to the integer number such that (see also Tab 1):

$$m^2(m-1)^2 \le K_C < m^2(m+1)^2$$

$K_C$ : $K_C = \dfrac{C_0 \ell^4}{\pi^4 E I_w} 10^6$

$C_0$ : Spring stiffness of the attached plating:

$$C_0 = \frac{E t_p^3}{2{,}73 s} 10^{-3}$$

### 2.2.4 Web buckling of axially loaded stiffeners

The Euler buckling stress of the stiffener web is obtained, in N/mm², from the following formulae:

- for flat bars:

$$\sigma_E = 16\left(\frac{t_W}{h_W}\right)^2 10^4$$

- for stiffeners with face plate:

$$\sigma_E = 78\left(\frac{t_W}{h_W}\right)^2 10^4$$

**Table 1 : Torsional buckling of axially loaded stiffeners Number m of half waves**

| $K_C$ | $0 \le K_C < 4$ | $4 \le K_C < 36$ | $36 \le K_C < 144$ |
|---|---|---|---|
| m | 1 | 2 | 3 |

## 2.3 Critical stress for aluminium members

### 2.3.1 General

The critical buckling stress is to be obtained, in N/mm², from the following formulae:

$$\sigma_c = \sigma_E \qquad \text{for } \sigma_E \le \frac{R'_{p0,2}}{2}$$

$$\sigma_c = R'_{p0,2}\left(1 - \frac{R'_{p0,2}}{4\sigma_E}\right) \qquad \text{for } \sigma_E > \frac{R'_{p0,2}}{2}$$

where:

$\sigma_E = \min(\sigma_{E1}, \sigma_{E2}, \sigma_{E3})$

$\sigma_{E1}$ : Euler column buckling stress, in N/mm², given in [2.2.2]

$\sigma_{E2}$ : Euler torsional buckling stress, in N/mm², given in [2.2.3]

$\sigma_{E3}$ : Euler web buckling stress, in N/mm², given in [2.2.4]

$R'_{p0,2}$ : Minimum guaranteed yield stress of the parent metal in welded condition, in N/mm², defined in Ch 4, Sec 3.

### 2.3.2 Column buckling of axially loaded stiffeners

The Euler column buckling stress is obtained, in N/mm², from the following formula:

$$\sigma_E = \pi^2 E \frac{I_e}{A_e \ell^2} 10^{-4}$$

### 2.3.3 Torsional buckling of axially loaded stiffeners

The Euler torsional buckling stresses is obtained, in N/mm², from the following formula:

$$\sigma_E = \frac{\pi^2 E I_w}{10^4 I_p \ell^2}\left(\frac{K_C}{m^2} + m^2\right) + 0{,}385 E \frac{I_t}{I_p}$$

where:

$I_w$ : Sectorial moment of inertia, in cm⁶, of the stiffener about its connection to the attached plating:

- for flat bars:

$$I_w = \frac{h_w^3 t_w^3}{36} 10^{-6}$$

- for T-sections:

$$I_w = \frac{t_f b_f^3 h_w^2}{12} 10^{-6}$$

- for angles and bulb sections:

$$I_w = \frac{b_f^3 h_w^2}{12(b_f + h_w)^2}[t_f b_f^2 + 2 b_f h_w + 4 h_w^2 + 3 t_w b_f h_w] 10^{-6}$$

$I_p$ : Polar moment of inertia, in cm⁴, of the stiffener about its connection to the attached plating:

- for flat bars:

$$I_p = \frac{h_w^3 t_w}{3} 10^{-4}$$

- for stiffeners with face plate:

$$I_p = \left(\frac{h_w^3 t_w}{3} + h_w^2 b_f t_f\right) 10^{-4}$$

$I_t$ : St. Venant's moment of inertia, in cm⁴, of the stiffener without attached plating:

- for flat bars:

$$I_t = \frac{h_w t_w^3}{3} 10^{-4}$$

- for stiffeners with face plate:

$$I_t = \frac{1}{3}\left[h_w t_w^3 + b_f t_f^3\left(1 - 0{,}63\frac{t_f}{b_f}\right)\right] 10^{-4}$$

m : Number of half waves, to be taken equal to the integer number such that (see also Tab 1):

$$m^2(m-1)^2 \le K_C < m^2(m+1)^2$$

$K_C$ : $K_C = \dfrac{C_0 \ell^4}{\pi^4 E I_w} 10^6$

$C_0$ : Spring stiffness of the attached plating:

$$C_0 = \frac{E t_p^3}{2{,}73 s} 10^{-3}$$

### 2.3.4 Web buckling of axially loaded stiffeners

The Euler buckling stress of the stiffener web is obtained, in N/mm², from the following formulae:

- for flat bars:

$$\sigma_E = 5{,}5\left(\frac{t_W}{h_W}\right)^2 10^4$$

- for stiffeners with face plate:

$$\sigma_E = 27\left(\frac{t_W}{h_W}\right)^2 10^4$$

## 2.4 Checking criteria

**2.4.1** As a rule, the normal stress considered for buckling check of stiffener is the stress determined by a direct calculation taking into account the global loads as defined in Part B, Chapter 6 and the strength characteristics of hull girder transverse sections as defined in Ch 4, Sec 4, [2].

### 2.4.2 Stiffeners parallel to the direction of compression

The critical buckling stress of the ordinary stiffener is to comply with the following formula:

$$\sigma_c \ge \sigma \cdot SF$$

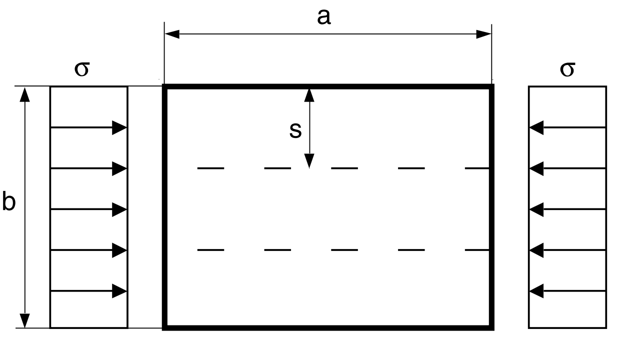
where:

$\sigma_c$ : Critical buckling stress, in N/mm², as calculated in [2.2.1] or [2.3.1]

$\sigma$ : Compression stress in the stiffener, in N/mm², as defined in [2.4.1]

SF : Safety factor as defined in Ch 4, Sec 3, Tab 2 or Ch 4, Sec 3, Tab 4.

Figure 17 : Buckling of stiffeners parallel to the direction of compression




## 3 Ordinary stiffeners sustaining lateral pressure

### 3.1 General: load point

#### 3.1.1 Lateral pressure for longitudinal stiffener

Unless otherwise specified, lateral pressure is to be calculated at mid-span of the ordinary stiffener considered.

#### 3.1.2 Lateral pressure for transversal stiffener

Unless otherwise specified, lateral pressure is to be calculated at the lower point and at the upper point of the ordinary stiffener considered.

### 3.2 Bending check

**3.2.1** As a rule, the design section modulus Z, in $cm^3$, of ordinary stiffeners sustaining lateral pressure is given by the formulae:

- for longitudinal stiffeners:

$$Z = 1000 \cdot coeff \cdot \frac{p \cdot s \cdot \ell^2}{m \cdot \sigma_{ad}}$$

where:

$\ell$ : Span of the siffener, in m, measured as indicated in Ch 4, Sec 4, [3]

s : Spacing between siffeners, in m,

p : Lateral pressure, in $kN/m^2$, as given

1) For hydrodynamic loads

a) For longitudinal stiffeners

$p = p_S$

as defined in Ch 7, Sec 1 taken into account [3.1.1]

b) For transversal stiffeners

$p = 3\ p_{S\ lower} + 2\ p_{S\ upper}$

where $p_{S\ lower}$ and $p_{S\ upper}$ are defined in Ch 7, Sec 1, taken into account [3.1.2]

2) For bottom slamming loads

$p = p_{sl}$ , as defined in Ch 7, Sec 2

3) For impact pressure on side shell

$p = p_{Smin}$ , as defined in Ch 7, Sec 1, [2.3]

$\sigma_{ad}$, $\tau_{ad}$ : Rule admissible stresses, in $N/mm^2$, depending on the type of materials and on the type of load (hydrodynamic load, slamming and impact load, test load or exceptional load in damage situation), defined in Ch 4, Sec 3, Tab 2 or Ch 4, Sec 3, Tab 44.

m : Coefficient depending on load type and/or end conditions :

- m = 60 for load type p as defined in 1) b)
- m = 12, 10 or 8 for types p as defined in 1) a), 2), 3), and depending on end conditions, as defined in Ch 4, Sec 4, [3.2].

coeff : Reduction coefficient equal to:

- $(1 - s / 2\ell) \geq 0$ in general case
- $(3\ell^2\text{-}0,36)\ 0,3/\ell^3$ for ordinary side shell stiffeners in the case where p is taken equal to the impact pressure on side shell $p_{Smin}$ as defined in Ch 7, Sec 1, [2.3.2] or Ch 7, Sec 1, [2.3.3], with $\ell$ being taken not less than 0,6m
- 1 for decks ordinary stiffeners.

### 3.3 Shearing check

**3.3.1** As a rule, the design shear area $A_{sh}$, in $cm^2$, of ordinary stiffeners sustaining lateral pressure is given by the formulae:

$$A_{sh} = 5 \cdot coeff \cdot \frac{p \cdot s \cdot \ell}{\tau_{ad}}$$

where:

s, $\ell$ : As indicated [3.2.1]

p : Lateral pressure, in $kN/m^2$, as given

1) For hydrodynamic loads

a) For longitudinal stiffeners

$p = p_S$

as defined in Ch 7, Sec 1 taken into account [3.1.1]

b) For transversal stiffeners

$p = (0,7\ p_{S\ lower} + 0,3\ p_{S\ upper})$

where $p_{S\ lower}$ and $p_{S\ upper}$ are defined in Ch 7, Sec 1, taken into account [3.1.2]

2) For bottom slamming loads

$p = p_{sl}$ , as defined in Ch 7, Sec 2

3) For impact pressure on side shell

$p = p_{Smin}$ , as defined in Ch 7, Sec 1, [2.3]

coeff : Reduction coefficient equal to:

- $(1 - s / 2\ell) \geq 0$ in general case
- $0,6/\ell$, without being taken superior to 1, for ordinary side shell stiffeners in the case where p is taken equal to the impact pressure on side shell $p_{s\ min}$ as defined in Ch 7, Sec 1, [2.3.2] or Ch 7, Sec 1, [2.3.3].
- 1 for decks ordinary stiffeners

$\tau_{ad}$ : Rule admissible shear stresse, in $N/mm^2$, as defined in Ch 4, Sec 3, Tab 2 or Ch 4, Sec 3, Tab 4 as appropriate.

# 4 Primary supporting members sustaining lateral pressure

## 4.1 General

**4.1.1** The primary supporting members are designed as indicated in [3] for ordinary stiffeners without taking into account the coefficients coeff and coeft, with lateral loads depending on primary member under consideration:

- bottom primary members: hydrodynamic loads as given in Ch 7, Sec 1, [2.2] and Ch 7, Sec 2, [2] or Ch 7, Sec 2, [3]
- side primary members: hydrodynamic loads as given in Ch 7, Sec 1, [2.2]
- decks primary members: minimum sea loads as given in Ch 7, Sec 1, [3.1] or Ch 7, Sec 1, [3.2].

For deck primary structure exposed to sea pressure, the section modulus Z and shear area $A_{sh}$ , calculated as indicated in [3], can be reduced by the following coefficients:

- 0,8 for primary structure of exposed supertructure decks
- $(1-0,05\ell) > 0,8$ for primary structure of exposed decks.

# 5 Curved primary supporting members

## 5.1 General

**5.1.1** The curvature of primary supporting members may be taken into account by direct analysis.

The curvate primary stiffener analysis can be carried out with Bureau Veritas program defined in Ch 1, Sec 4.

### 5.1.2 Model principles

In case of 2-D or 3-D beam structural model, the curved primary supporting members are to be represented by a number N of straight beams, N being adequately selected to minimize the spring effect in way of knuckles.

The stiffness of knuckles equivalent springs is considered as unaffecting the local bending moment and shear forces distribution where the angle between two successive beams is not more than 3°.