PLATE GIRDER BRIDGE DESIGN TO THE EUROCODES

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LEARNING OUTCOMES

At the end of this workshop, participants should:

1. Be able to discuss the conceptual difference between UB and fabricated plate girder

2. Be able to explain the failure modes of restrained plate girder.

3. Be able to analyze and design plate girder for shear buckling and moment capacity

LEARNING OUTCOMES

At the end of this workshop, participants should:

4. Be able to analyse and design intermediate stiffener in plate girder for tension field action (TFA).

5. Be able to discuss the difference between bearing stiffeners and intermediate stiffener.

6. Be able to carry out detailing of stiffeners.

LECTURE 1:
PLATE GIRDER BEHAVIOUR AND BASIC DIMENSIONING

1. General consideration
2. Plate girder design rules
3. Influence of buckling on design
4. Post-buckling strength of web
5. Concluding summary

1.0 GENERAL CONSIDERATION

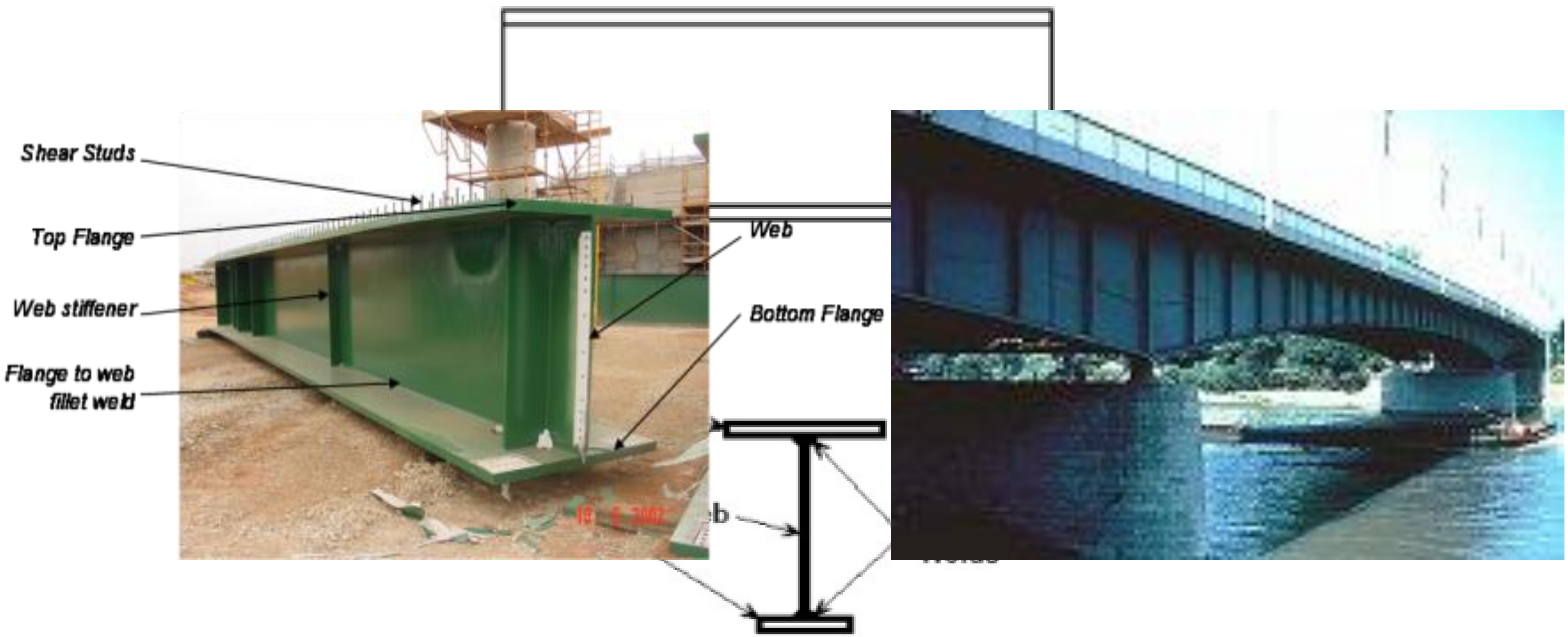


Figure 1: Plate girder overview

1.0 GENERAL CONSIDERATION

- Plate girders are fabricated by welding flanges to a web plate as shown in Figure 1.
- The flanges are generally significantly thicker than the web because of the lower buckling capability of one edge unsupported plates.
- If the girder is to be used compositely with a concrete deck, the top flange will generally be narrower and will carry shear connectors for composite action with the concrete slab. In such a case the width of the flange has to be sufficient for the construction condition carrying the wet concrete before composite action is achieved.

1.0 GENERAL CONSIDERATION

- Cross girders may be welded or bolted between adjacent girders, for example for a rail bridge where they support the deck that carries the track.
- In all cases the main girders need to be appropriately designed to resist lateral torsional buckling. In the former case this can only occur during construction as the concrete deck, once acting compositely, will provide full restraint to the concrete top flange.

1.0 GENERAL CONSIDERATION

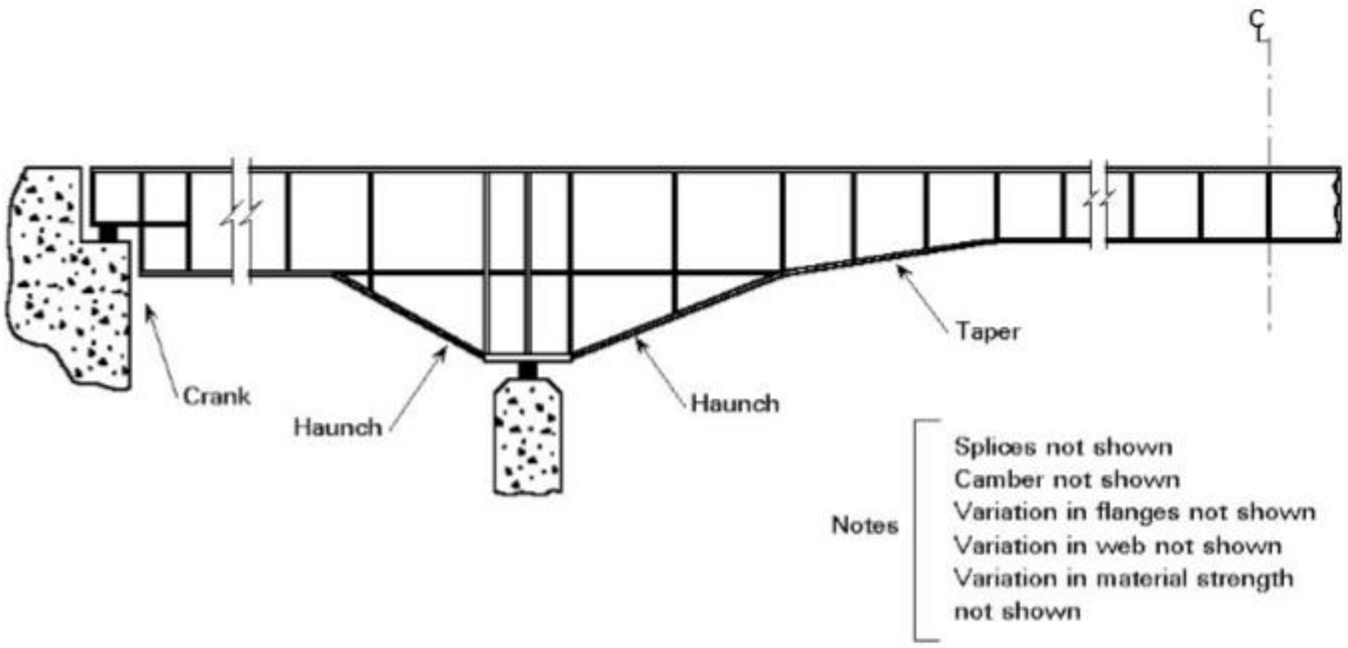
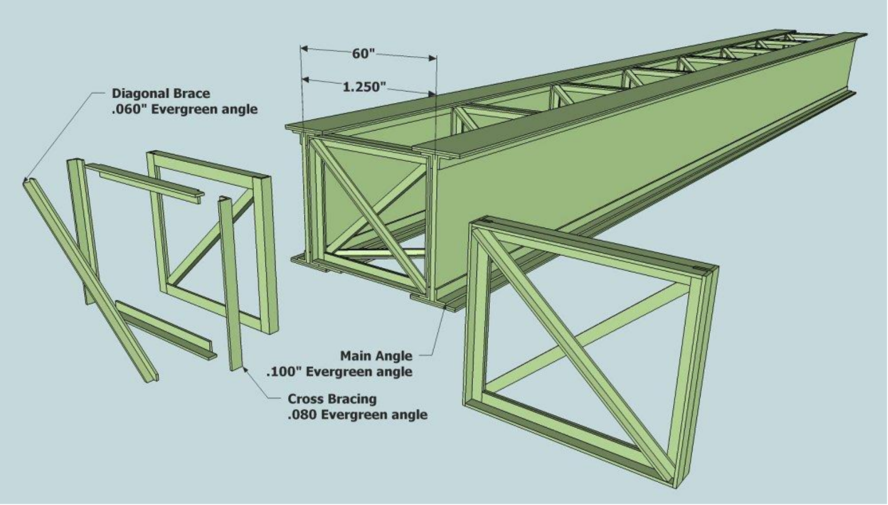


Figure 2: Longitudinal view

1.0 GENERAL CONSIDERATION



1.0 GENERAL CONSIDERATION

- During construction, restraint is normally proved by cross bracing at appropriate intervals bolted to outstands welded to the girders. Where the girders are connected by cross beams near the bottom of the main girders, V-frame action whereby the cross beams provide a moment restraint stiffness can be used to prevent torsional buckling of the girders.
- Because they can be fabricated from plates of any width and thickness, plate girders can be used for much longer spans than using beams from hot rolled sections with their restricted availability (See Figure 2)
- Indeed spans of over 200m are possible, often with haunches provided near continuous supports to increase moment capacity.

1.0 GENERAL CONSIDERATION

- Fabrication can be particularly effective where stiffening is kept to a minimum and welding carried out automatically.
- Girders are generally relatively deep to provide the moment resistance and webs therefore generally need only to be thin to resist the applied shear.
- This leads to relatively thin web design. In contrast there is little benefit in having thin flanges so these are designed to reduce buckling problems.

1.0 GENERAL CONSIDERATION

- This tends to lead to webs which require stiffening, at least over the supports to prevent crippling caused by the high point loads and to allow the shear in the web to be transferred as compression down the bearing stiffeners into the support.
- Transverse stiffeners are also often provided for longer spans to enhance the buckling capacity of the web, principally by increasing its shear capability.
- In a minority of cases longitudinal web stiffeners are also provided to increase the web buckling capacity by reducing the slenderness of the panels within the depth of the girder as shown in Figure.

1.0 GENERAL CONSIDERATION

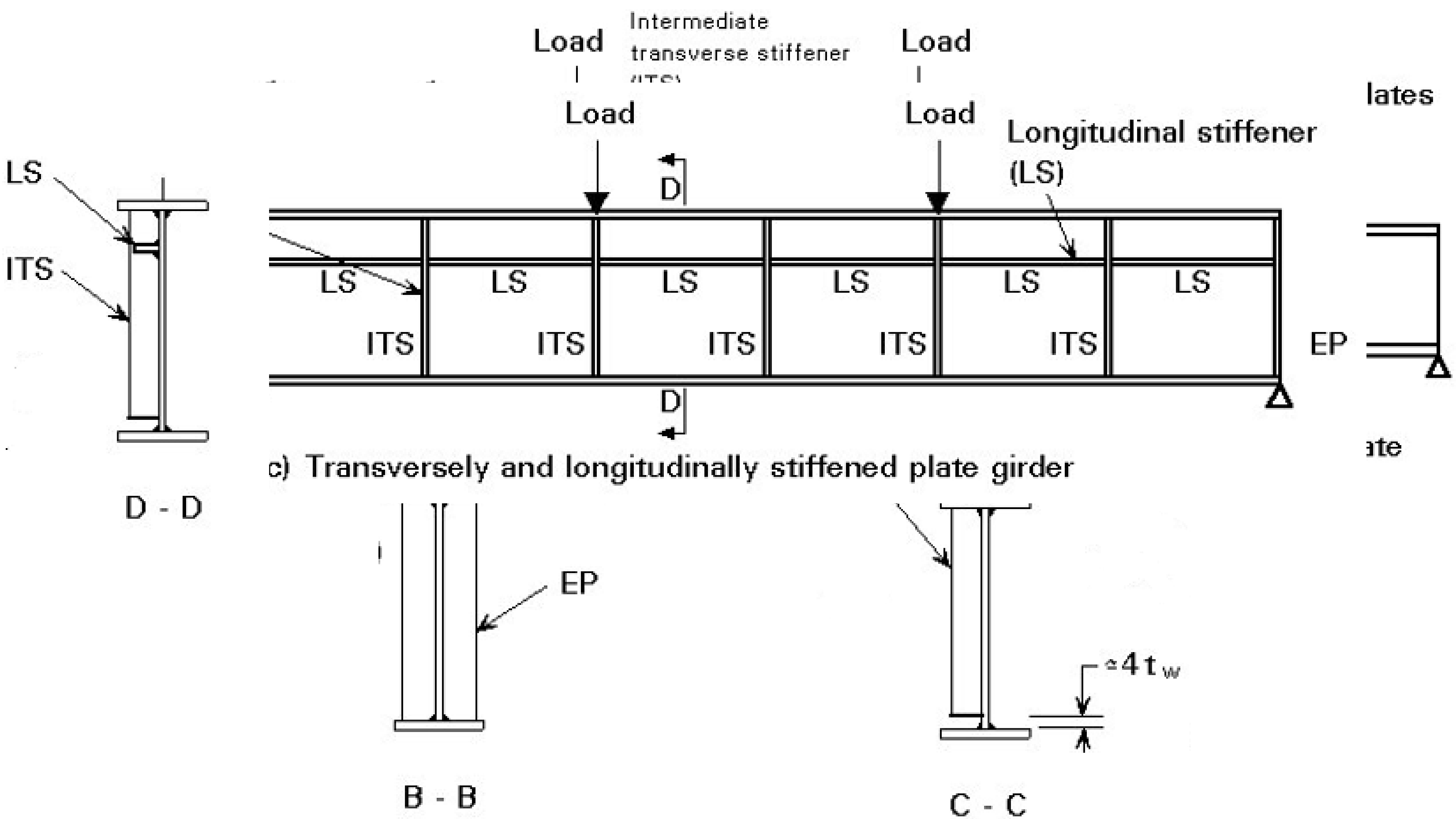


Figure 3: Types of plate girder

1.0 GENERAL CONSIDERATION

- However, this should be considered carefully as it significantly increases fabrication complexity both by preventing the use of automatic welding and also by introducing a significant number of complex cutting and welding operations at the connection between transverse and longitudinal stiffeners.
- It is worth noting that intermediate transverse stiffeners are often curtailed short of the tension flange in order to provide better fatigue resistance (see Figure 4).
- Such curtailment does not affect the buckling enhancement of the stiffening as the latter still provides out-ofplane bending support to the web plate.

1.0 GENERAL CONSIDERATION

- The Eurocode requires stiffeners to be spaced at between one and three times the web depth.
- While BS5400 Part 3 imposes no such restriction it is a sensible range for normal design application. If the stiffener spacing were any closer it is likely that increased fabrication cost would more than offset the savings in web material achieved through using a thinner web and if more widely spaced the enhancement to web buckling capacity would be limited.

Curtailment of transverse web stiffeners to improve fatigue response.

Figure 4: Curtailment of transverse web stiffeners

1.0 GENERAL CONSIDERATION

- In longer span girders it is possible to vary the cross section in the longitudinal direction to match more closely the variation of moment and / shear along the span.
- Flange thickness can be varied with full strength butt welding providing a smooth external flange surface. Flange width can also be varied (see Figure 5)
- The possibility of a haunched girder has already been mentioned. Rather than vary the web thickness, it is more normal to vary transverse stiffener spacing or introduce a longitudinal stiffener over part of the span, for example in the compression region over a continuous support.

TYPES

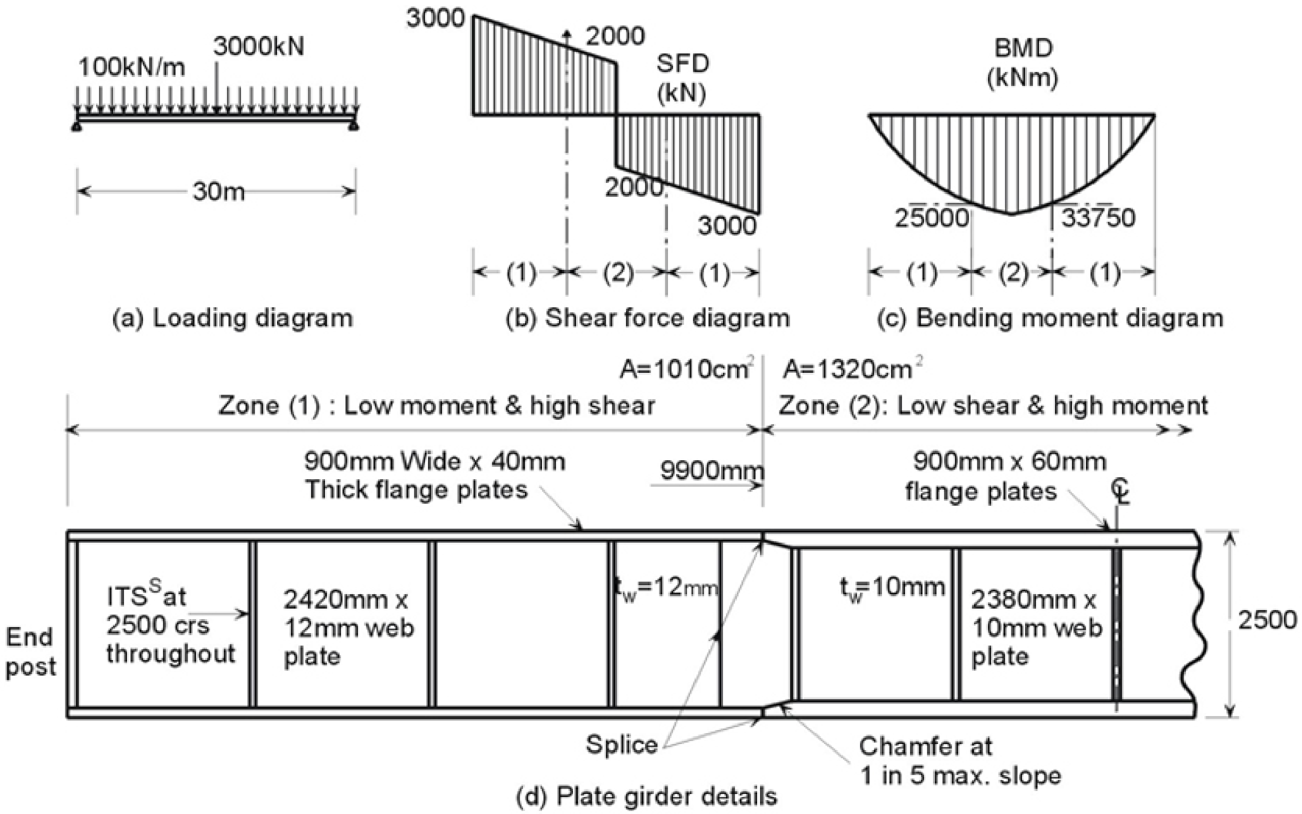


Figure 5: Distribution of loading, shear force and bending moment

1.0 GENERAL CONSIDERATION

- With modern fabrication it is also possible to use different yield strength steels for different sections along the bridge to achieve a variation in capability.
- In all cases the benefit of better matching the resistance of the structure to the applied loading needs to be weighed against any increased fabrication complexity and hence cost that might ensue.

1.0 GENERAL CONSIDERATION

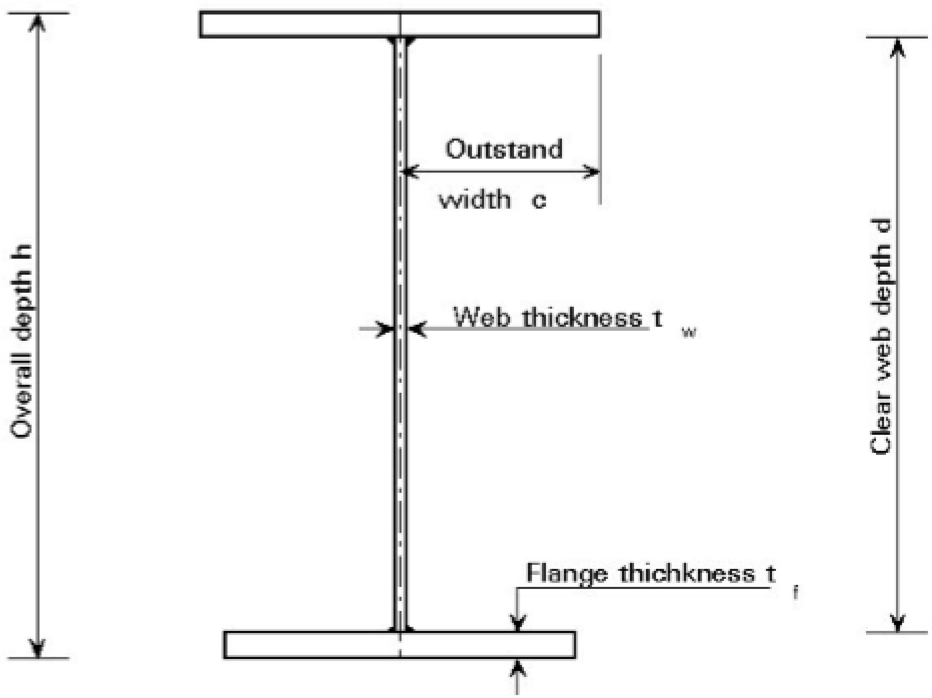
1.0 GENERAL CONSIDERATION

- It is possible to define a range of practical dimensional proportions that are typical of bridge construction.
- Plate girders with spans up to around a 1000 metres have been used as suspension structures but one notorious example, the Tacoma Narrows bridge with a span of 853 metres, failed disastrously because of its flexibility to wind excitation in 1940.
- Box girders are now generally accepted as more appropriate for the longer spans because of their inherent torsional stiffness.

1.0 GENERAL CONSIDERATION

- More modest plate girder suspended spans of up to 400 metres are not unusual. Cable stayed composite plate girder bridges have been constructed with spans up to about 500 metres.
- Composite plate girders without additional support are used for many bridge structures over a range of more modest spans as a competitor to prestressed concrete structures.

PROPORTIONS



PROPORTIONS

- Overall girder depths range between one tenth to one twentieth of the span with the larger vales used for longer spans.
- Flange widths will tend to be between a third and a fifth of the girder depth. As has been noted previously flange plates are usually designed to be stocky to preclude loss of strength by buckling.
- They would normally be designed as at least semi-compact, a definition for slenderness used in Eurocode 3 although not in BS5400 Part 3, which restricts section classifications to compact and non-compact.

PROPORTIONS

- In terms of the Eurocode this means applying a limit of 14 t_f E to the flange outstand width where t_f is the flange thickness and E is (235/E).
- The web thickness cannot be simply defined in terms of a standard range. Because of the range of stiffening options a web might have a depth to thickness ratio ranging from between 80 and 500.
- Eurocode requires stiffeners to be spaced at between one and three times the web depth. While BS5400 Part 3 imposes no such restriction it is a sensible range for normal design application
- Longitudinal stiffening can be considered for webs with slenderness ratios larger than about 200.

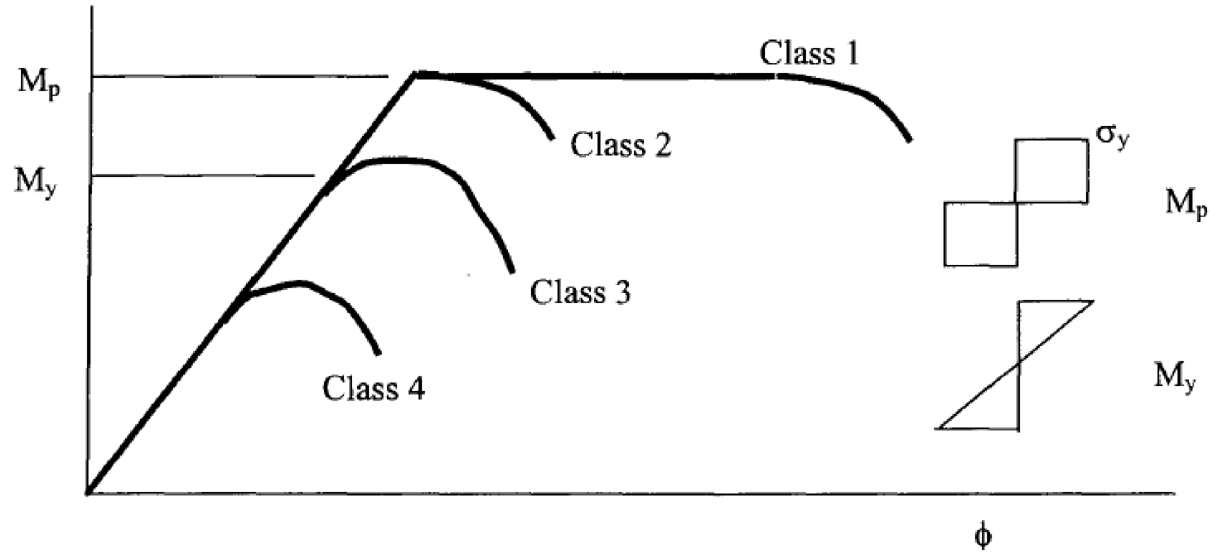
PROPORTIONS

- The terms compact and semi-compact and non - compact define the moment that a girder can carry prior to buckling and also its ability to redistribute moment along the span prior to ultimate collapse.
- In Eurocodes there are four classes of section (refer Figure 5).
- Class 1 sections, generally called compact are able to reach their plastic moment Mp value and to attain sufficient rotation prior to buckling to allow redistribution of moments and hence the use of plastic collapse analysis.
- Class 2 sections, compact, are also able to reach the Mp value but have limited rotational capacity.

PROPORTIONS

- Class 3 sections, semi-compact, are able to reach their first yield moment My (a moment which at least achieves the extreme fibre of the girder) prior to buckling.
- Class 4 sections, slender girders (the equivalent of non-compact in BS5400) will not reach My and require design rules which either define a limit stress lower than the yield stress or which define a reduced effective section either through an effective width concept or by using an artificial reduced thickness.

PROPORTIONS



Moment rotation behaviour of girders with different classes of cross section.

Figure 5: Classification of cross-section

DESIGN CONCEPT

The principal functions of the main components found in plate girders may be summarised as follows:

- ❑ Flanges resist moment
- ❑ Web resists shear
- ❑ Web/flange welds resist longitudinal shear at interface
- ❑ Vertical stiffeners improve shear buckling resistance
- ❑ Longitudinal stiffeners improve shear and/or bending resistance

PLATE GIRDER DESIGN RULE

- Designing plate girder (which is permitted by Eurocode 3) assuming
  - the flanges to carry all the moment
  - the web taking all the shear
- The required flange plate area can be calculated as follows;

$M_{f.Rd} = b_f\, t_f\, (h - t_f) f_y / \gamma_{MO}$ ……………………………………………(1)

$A_f = M_{ED}/[(h - t_f) f_y / \gamma_{MO}$ …………………………………………………(2)

The minimum web thickness is control by flange induce buckling, whereby;

PLATE GIRDER DESIGN RULE

- A simple and often effective design basis for a girder is to design the flange to carry all the moment and the web to carry all the shear.
- Even where a more complex basis is used for the final design, this can be a very effective method for initial sizing.
- This approach recognizes the inherent capabilities of the two elements.

Minimum Web Thickness

Cl 8, EC3-1-5

The web should be sized to avoid the flange undergoing local buckling due to the web being unable to support the flange, which is known as flange induced buckling.

$$\frac{h_w}{t_w} \leq k \frac{E}{f_{yf}} \sqrt{\frac{A_w}{A_c}}$$

f_{yf} = yield strength of the compression flange
A_c = the effective area of the compression flange
A_w = area of the web
k = 0.3 where plastic hinge rotation is utilized, 0.4 if the plastic resistance is utilized, 0.55 if the elastic resistance is utilized.

Bending Resistance

The section classification is determined similar to rolled sections. Two methods that can be used for restrained compression flange design:

1. The flanges carrying the bending moment and the web carries the shear force
   – best used when the maximum bending moment and maximum shear force are not coincident.
   – the moment capacity is only dependant on the section classification of the flanges as the web does not carry compression action.

2. The girder carrying the force as an entity
   – more complex method and may not show any resultant economies over the first method but should be utilized when maximum moment and maximum shear are co-incident.

Basic Dimensioning

Method 1:

Assuming the moment to be resisted by the flanges alone, then

$$M_{\mathrm{Rd}} = f_{\mathrm{yd}} b_{\mathrm{f}} t_{\mathrm{f}} h_{\mathrm{w}} \tag{5.111}$$

where f_{yd} is the design strength of the flanges, t_{f} and b_{f} the thickness and width of the flange plates and h_{w} the distance between the internal faces of the flanges. Equation (5.111) is slightly conservative for beams of Classes 1 to 3.

The cross-sectional area A is given by

$$A = 2b_{\mathrm{f}} t_{\mathrm{f}} + h_{\mathrm{w}} t \tag{5.112}$$

Eliminate $b_{\mathrm{f}} t_{\mathrm{f}}$ between Eqs (5.111) and (5.112) to give

$$A = \frac{2M_{\mathrm{Rd}}}{h_{\mathrm{w}} f_{\mathrm{yd}}} + h_{\mathrm{w}} t \tag{5.113}$$

Basic Dimensioning

Method 1:

Define the web slenderness ratio h_w/t as λ, then Eq. (5.113) becomes

$$A = \frac{2M_{\mathrm{Rd}}}{\lambda t f_{\mathrm{yd}}} + \lambda t^2 \tag{5.114}$$

For an optimum solution, $\mathrm{d}A/\mathrm{d}t = 0$, so Eq. (5.114) becomes,

$$\frac{\mathrm{d}A}{\mathrm{d}t} = -\frac{2M_{\mathrm{Rd}}}{\lambda f_{\mathrm{yd}} t^2} + 2\lambda t = 0 \tag{5.115}$$

or,

$$t = \sqrt[3]{\frac{M_{\mathrm{Rd}}}{\lambda^2 f_{\mathrm{yd}}}} \tag{5.116}$$

Basic Dimensioning

Method 1:

$$h_w = \sqrt[3]{\frac{\lambda M_{Rd}}{f_{yd}}} \tag{5.117}$$

The area of the web, A_w is then given by

$$A_w = \sqrt[3]{\frac{M_{Rd}^2}{\lambda f_{yd}^2}} \tag{5.118}$$

Using Eq. (5.112), the flange area, A_f is given by

$$A_f = b_f t_f = \frac{M_{Rd}}{f_{yd}\sqrt[3]{\frac{\lambda M_{Rd}}{f_{yd}}}} = \sqrt[3]{\frac{M_{Rd}^2}{\lambda f_{yd}^2}} \tag{5.119}$$

Thus the area of a single flange is equal to that of the web.

Basic Dimensioning

Method 2 (Classes 1 and 2):

It is recognized that this is not a likely case but is included for completeness. The moment is resisted by the complete section, when the moment capacity is given by that due to the flanges (Eq. (5.111)) and the additional plastic capacity of the web

$$M_{Rd} = f_{yd} b_f t_f h_w + f_{yd} \frac{t h_w^2}{4} \tag{5.120}$$

where f_{yd} is the design strength of the flanges, t_f and b_f the thickness and width of the flange plates and h_w the distance between the internal faces of the flanges.

The cross-sectional area A is given by Eqs (5.112), thus from Eqs (5.112) and (5.120), $b_f t_f$ is given by

$$b_f t_f = \frac{M_{Rd}}{f_{yd} h_w} - \frac{h_w t}{4} \tag{5.121}$$

Basic Dimensioning

Method 2 (Classes 1 and 2):

Eliminate $b_f t_f$ between Eqs (5.112) and (5.121) to give

$$A = \frac{2M_{Rd}}{h_w f_{yd}} + h_w t - 2\frac{h_w t}{4} = \frac{2M_{Rd}}{h_w f_{yd}} + \frac{h_w t}{2} \tag{5.122}$$

Define the web slenderness ratio h_w/t as λ, then Eq. (5.122) becomes

$$A = \frac{2M_{Rd}}{\lambda t f_{yd}} + \frac{\lambda t^2}{2} \tag{5.123}$$

For an optimum solution, $dA/dt = 0$, so Eq. (5.123) becomes,

$$\frac{dA}{dt} = -\frac{2M_{Rd}}{\lambda f_{yd} t^2} + \lambda t = 0 \tag{5.124}$$

Basic Dimensioning

Method 2 (Classes 1 and 2):

or,

$$t = \sqrt[3]{\frac{2M_{\mathrm{Rd}}}{\lambda^2 f_{\mathrm{yd}}}} \tag{5.125}$$

and

$$h_{\mathrm{w}} = \sqrt[3]{\frac{2\lambda M_{\mathrm{Rd}}}{f_{\mathrm{yd}}}} \tag{5.126}$$

The area of the web, A_{w} is then given by

$$A_{\mathrm{w}} = \sqrt[3]{\frac{4M_{\mathrm{Rd}}^2}{\lambda f_{\mathrm{yd}}^2}} \tag{5.127}$$

Basic Dimensioning

Method 2 (Classes 1 and 2):

Using Eq. (5.121), the flange area, A_f is given by

$$A_f = b_f t_f = \sqrt[3]{\frac{M_{Rd}^2}{\lambda f_{yd}^2}}\left[\sqrt[3]{\frac{1}{2}} - \sqrt[3]{\frac{1}{16}}\right] \qquad (5.128)$$

or,

$$\frac{A_f}{A_w} = \frac{\sqrt[3]{\frac{M_{Rd}^2}{\lambda f_{yd}^2}}\left[\sqrt[3]{\frac{1}{2}} - \sqrt[3]{\frac{1}{16}}\right]}{\sqrt[3]{\frac{4M_{Rd}^2}{\lambda f_{yd}^2}}} = \frac{1}{4} \qquad (5.129)$$

Thus the area of the web is equal four times that of a single flange.

Basic Dimensioning

Method 2 (Classes 3):

The moment is resisted by the complete section, when the moment capacity is given by that due to the flanges (Eq. (5.111)) and the additional elastic capacity of the web

$$M_{\mathrm{Rd}} = f_{\mathrm{yd}} b_{\mathrm{f}} t_{\mathrm{f}} h_{\mathrm{w}} + f_{\mathrm{yd}} \frac{t h_{\mathrm{w}}^2}{6} \tag{5.130}$$

where f_{yd} is the design strength of the flanges, t_{f} and b_{f} the thickness and width of the flange plates and h_{w} the distance between the internal faces of the flanges.

The cross-sectional area A is given by Eq. (5.112), thus from Eqs (5.112) and (5.130), $b_{\mathrm{f}} t_{\mathrm{f}}$ is given by

$$b_{\mathrm{f}} t_{\mathrm{f}} = \frac{M_{\mathrm{Rd}}}{f_{\mathrm{yd}} h_{\mathrm{w}}} - \frac{h_{\mathrm{w}} t}{6} \tag{5.131}$$

Basic Dimensioning

Method 2 (Classes 3):

Eliminate $b_f t_f$ between Eqs (5.112) and (5.131) to give

$$A = \frac{2M_{Rd}}{h_w f_{yd}} + h_w t - 2\frac{h_w t}{6} = \frac{2M_{Rd}}{h_w f_{yd}} + \frac{2h_w t}{3} \tag{5.132}$$

Define the web slenderness ratio h_w/t as λ, then Eq. (5.132) becomes

$$A = \frac{2M_{Rd}}{\lambda t f_{yd}} + \frac{2\lambda t^2}{3} \tag{5.133}$$

For an optimum solution, $dA/dt = 0$, so Eq. (5.133) becomes,

$$\frac{dA}{dt} = -\frac{2M_{Rd}}{\lambda f_{yd} t^2} + \frac{4}{3}\lambda t = 0 \tag{5.134}$$

Basic Dimensioning

Method 2 (Classes 3):

or,

$$t = \sqrt[3]{\frac{3M_{\mathrm{Rd}}}{2\lambda^2 f_{\mathrm{yd}}}} \tag{5.135}$$

and

$$h_{\mathrm{w}} = \sqrt[3]{\frac{3\lambda M_{\mathrm{Rd}}}{2f_{\mathrm{yd}}}} \tag{5.136}$$

The area of the web, A_{w} is then given by

$$A_{\mathrm{w}} = \sqrt[3]{\frac{9M_{\mathrm{Rd}}^2}{4\lambda f_{\mathrm{yd}}^2}} \tag{5.137}$$

Basic Dimensioning

Method 2 (Classes 3):

Using Eq. (5.131), the flange area, A_f is given by

$$A_f = b_f t_f = \sqrt[3]{\frac{M_{Rd}^2}{\lambda f_{yd}^2}}\left[\sqrt[3]{\frac{2}{3}} - \frac{1}{6}\sqrt[3]{\frac{9}{4}}\right] \quad (5.138)$$

Thus

$$\frac{A_f}{A_w} = \frac{\sqrt[3]{\frac{M_{Rd}^2}{\lambda f_{yd}^2}}\left[\sqrt[3]{\frac{2}{3}} - \frac{1}{6}\sqrt[3]{\frac{9}{4}}\right]}{\sqrt[3]{\frac{9M_{Rd}^2}{4\lambda f_{yd}}}} = \frac{1}{2} \quad (5.139)$$

Thus the area of the web is equal twice that of a single flange.

INFLUENCE OF BUCKLING ON DESIGN

1. Shear buckling of web

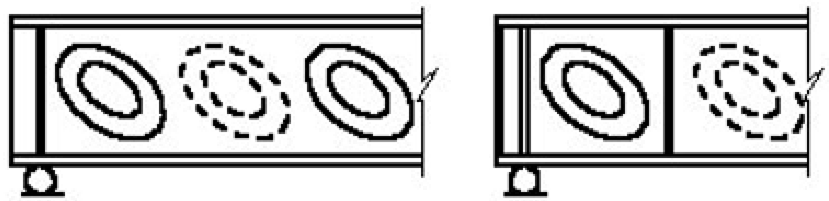
Buckling type	Illustration
Shear buckling of web	

Figure 6: Shear buckling of web

INFLUENCE OF BUCKLING ON DESIGN

- Once the h_w/t_w value for an unstiffened web exceeds a limiting figure (69ε in Eurocode 3) the web will buckle in shear before it reaches its full shear capacity $A_w\tau_y$.
- Diagonal buckles, of the type shown in Figure, resulting from the diagonal compression associated with the web shear will form.
- Their appearance may be delayed through the use of vertical

stiffeners, see Figure since the load at which shear buckling is initiated is a function of both d/tw and panel aspect ratio a/d.

INFLUENCE OF BUCKLING ON DESIGN

2. Lateral Torsional Buckling (LTB) of girder

Buckling type	Illustration
Lateral-torsional buckling of girder	

Figure 7: Lateral Torsional Buckling

- Lateral torsional buckling occurs because the low torsional and transverse stiffness of the girder, compared to the main vertical stiffness, allow failure by sideways and twisting deformation of the girder even when loaded vertically.
- The section will be particularly prone to this form of failure if the vertical loading is applied to the compression flange providing an increasing eccentricity of load as the girder deflects sideways.
- It can be prevented through the measures discussed earlier by preventing sideways movement of the compression flange either directly through bracing or indirectly through the V-frame action.

INFLUENCE OF BUCKLING ON DESIGN

3. Local buckling of compression flange

Buckling type	Illustration
Local buckling of compression flange	

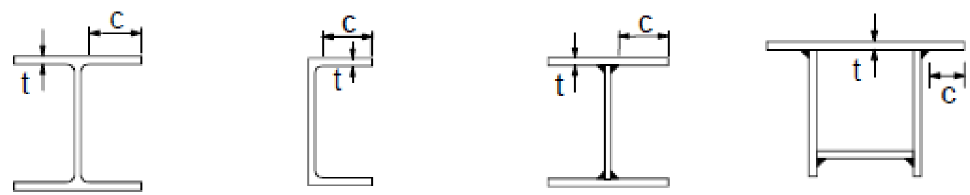
Figure 8: Local Buckling of Compression Flange

INFLUENCE OF BUCKLING ON DESIGN

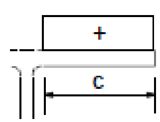
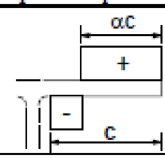
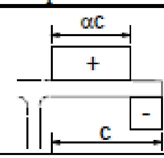
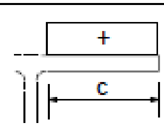
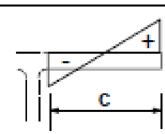
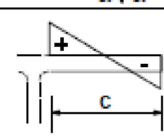
Provided that outstand proportions c/t_f are suitably restricted, local buckling will have no effect on the girder's load carrying resistance.

Table 5.2 (sheet 2 of 3): Maximum width-to-thickness ratios for compression parts

Outstand flanges



Rolled sections — Welded sections

Class	Part subject to compression	Part subject to bending and compression: Tip in compression	Part subject to bending and compression: Tip in tension
Stress distribution in parts (compression positive)			
1	$c/t \le 9\varepsilon$	$c/t \le \dfrac{9\varepsilon}{\alpha}$	$c/t \le \dfrac{9\varepsilon}{\alpha\sqrt{\alpha}}$
2	$c/t \le 10\varepsilon$	$c/t \le \dfrac{10\varepsilon}{\alpha}$	$c/t \le \dfrac{10\varepsilon}{\alpha\sqrt{\alpha}}$
Stress distribution in parts (compression positive)			
3	$c/t \le 14\varepsilon$	$c/t \le 21\varepsilon\sqrt{k_\sigma}$	

INFLUENCE OF BUCKLING ON DESIGN

4. Compression buckling of web

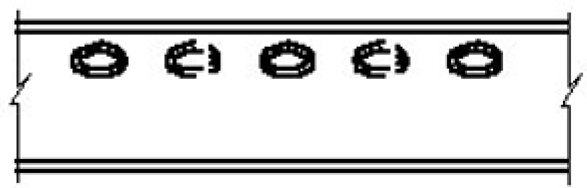
Buckling type	Illustration
Compression buckling of web	

Figure 9: Compression buckling of web

INFLUENCE OF BUCKLING ON DESIGN

- Webs for which $h_w/t_w \leq 124\varepsilon$ and which are not subject to any axial load will permit the full elastic moment resistances of the girder to be attained. If this limit of h_w/t_w is exceeded, then moment resistance must be reduced accordingly. If it is desired to reach the girder's full plastic moment resistance a stricter limit will be appropriate.

INFLUENCE OF BUCKLING ON DESIGN

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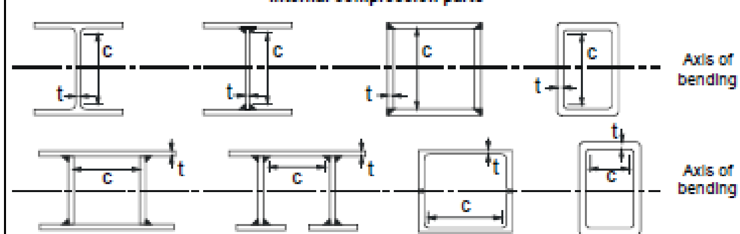
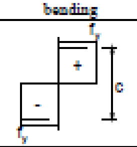
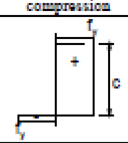
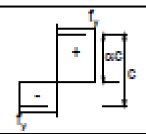
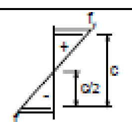
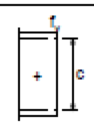
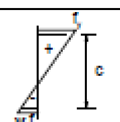
Table 5.2 (sheet 1 of 3): Maximum width-to-thickness ratios for compression parts

Internal compression parts



Class	Part subject to bending	Part subject to compression	Part subject to bending and compression
Stress distribution in parts (compression positive)			
1	$c/t \le 72\varepsilon$	$c/t \le 33\varepsilon$	when $\alpha > 0,5$: $c/t \le \frac{396\varepsilon}{13\alpha - 1}$ when $\alpha \le 0,5$: $c/t \le \frac{36\varepsilon}{\alpha}$
2	$c/t \le 83\varepsilon$	$c/t \le 38\varepsilon$	when $\alpha > 0,5$: $c/t \le \frac{456\varepsilon}{13\alpha - 1}$ when $\alpha \le 0,5$: $c/t \le \frac{41,5\varepsilon}{\alpha}$
Stress distribution in parts (compression positive)			
3	$c/t \le 124\varepsilon$	$c/t \le 42\varepsilon$	when $\psi > -1$: $c/t \le \frac{42\varepsilon}{0,67 + 0,33\psi}$ when $\psi \le -1$*): $c/t \le 62\varepsilon(1-\psi)\sqrt{(-\psi)}$

$\varepsilon = \sqrt{235/f_y}$	f_y	235	275	355	420	460
	ε	1,00	0,92	0,81	0,75	0,71

*) $\psi \le -1$ applies where either the compression stress $\sigma \le f_y$ or the tensile strain $\varepsilon_y > f_y/E$

INFLUENCE OF BUCKLING ON DESIGN

6. Flange induced buckling of the web

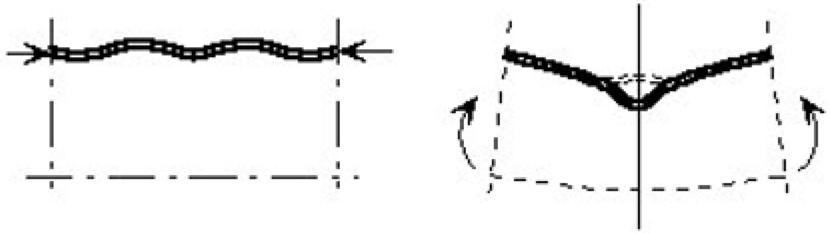
Buckling type	Illustration
Flange induced buckling of the web	

Figure 10: Flange induced buckling of the web

INFLUENCE OF BUCKLING ON DESIGN

If particularly slender webs are used, the compression flange may not receive enough support to prevent it from buckling vertically rather like an isolated strut buckling about its minor axis. This possibility may be eliminated by placing a suitable limit on h_w/t_w. Transverse stiffeners also assist in resisting this form of buckling.

INFLUENCE OF BUCKLING ON DESIGN

5. Local buckling of web (due to vertical load)

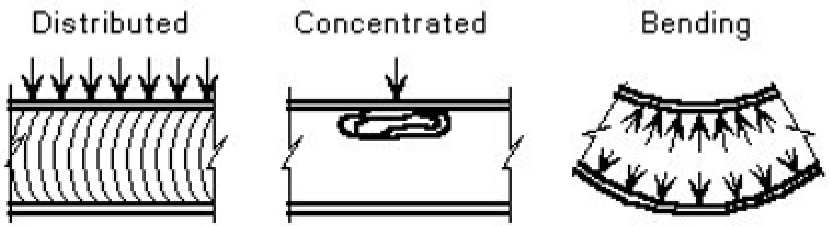
Buckling type	Illustration
Local buckling of web (due to vertical load)	

Figure 11: Local buckling of the web

INFLUENCE OF BUCKLING ON DESIGN

- Vertical loads may cause buckling of the web in the region directly under the load as for a vertical strut.
- The level of loading that may safely be carried before this happens will depend upon the exact way in which the load is transmitted to the web, the web proportions, and the level of overall bending present.

POST BUCKLING OF WEB

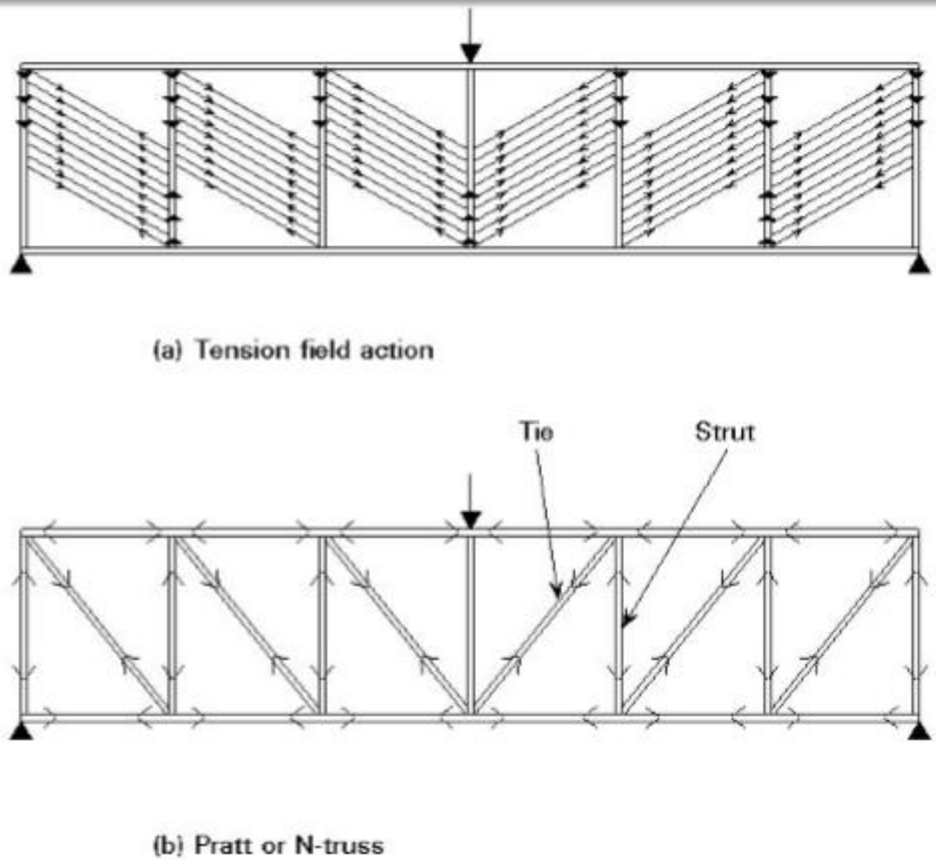


Figure 12: Post buckling of the web

POST BUCKLING OF WEB

- Owing to the post-buckling behaviour plates, unlike struts, are often able to support loads considerably in excess of their initial buckling load.
- In plate girder webs a special form of post-buckling termed "tension field action" is possible. Tension field action involves a change in the way in which the girder resists shear loading from the development of uniform shear in the web at low shear loads, to the equivalent truss action, shown in Figure 5, at much higher loads.

POST BUCKLING OF WEB

- In this action the elements equivalent to truss members are: the flanges, which form the chords; the vertical stiffeners which form the struts; and the diagonal tension bands which form the ties.
- The compressive resistance of the other diagonal of each web panel is virtually eliminated by the shear buckling.

CONCLUDING SUMMARY

- ✓ The main components in a plate girder have been identified and their principal functions noted.
- ✓ Initial sizing may be made on the basis that the flanges carry all of the moment and the web takes all of the shear.
- ✓ Shear buckling is likely to prevent the full web shear resistance from being attained in slender webs. Its appearance need not imply failure since additional load may be carried through tension field action.
- ✓ Web stiffeners (transverse and/or longitudinal) enhance both initial buckling and post-buckling resistance

GUIDED TUTORIAL 1 (30 MINUTES)

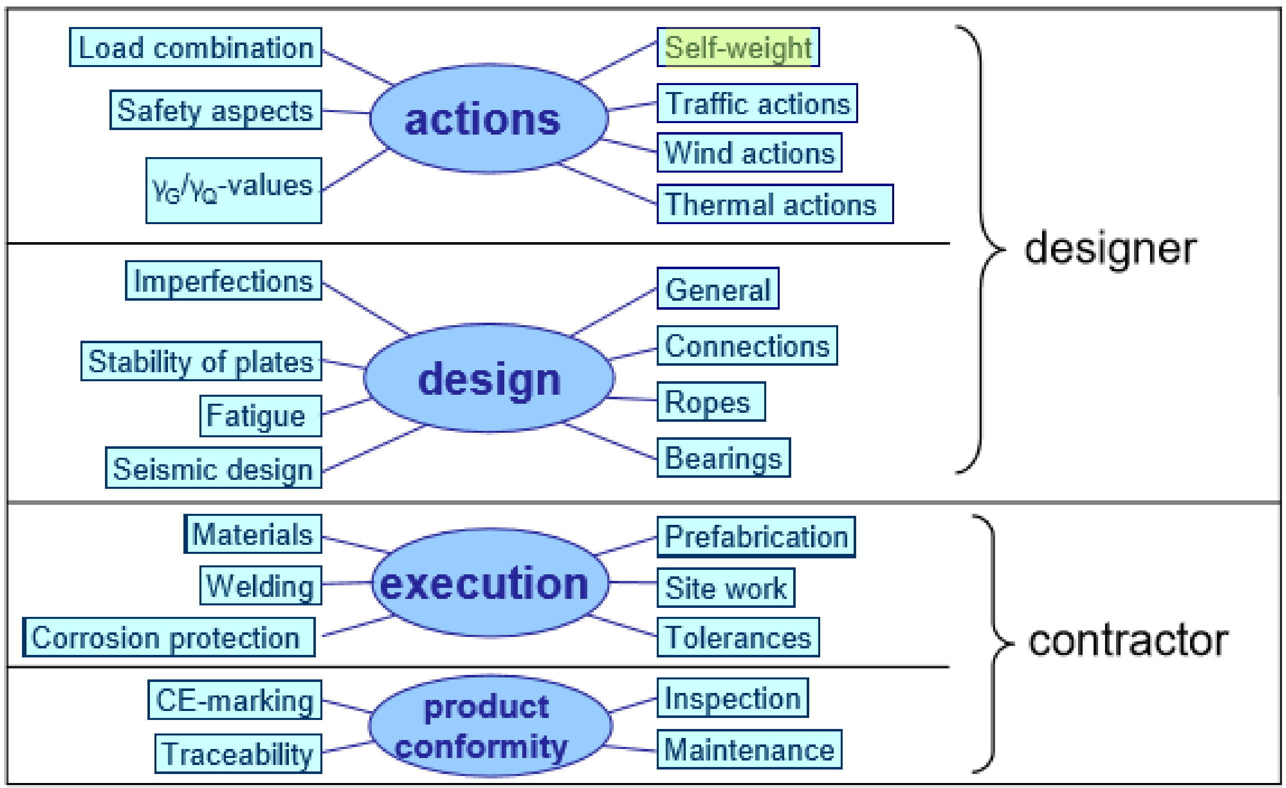
Proposed a suitable size of plate girder in grade S355 steel to carry a characteristic variable actions of 150 kN/m over a span of 20m. The compression flange is fully restrained against Lateral Torsional Buckling (LTB).

LECTURE 2:
DESIGN OF STEEL PLATE GIRDER BRIDGES

- General
- Basis of design
- Properties of material
- Classification of cross-section
- Resistance of plate girder cross-section
- Bolted connections

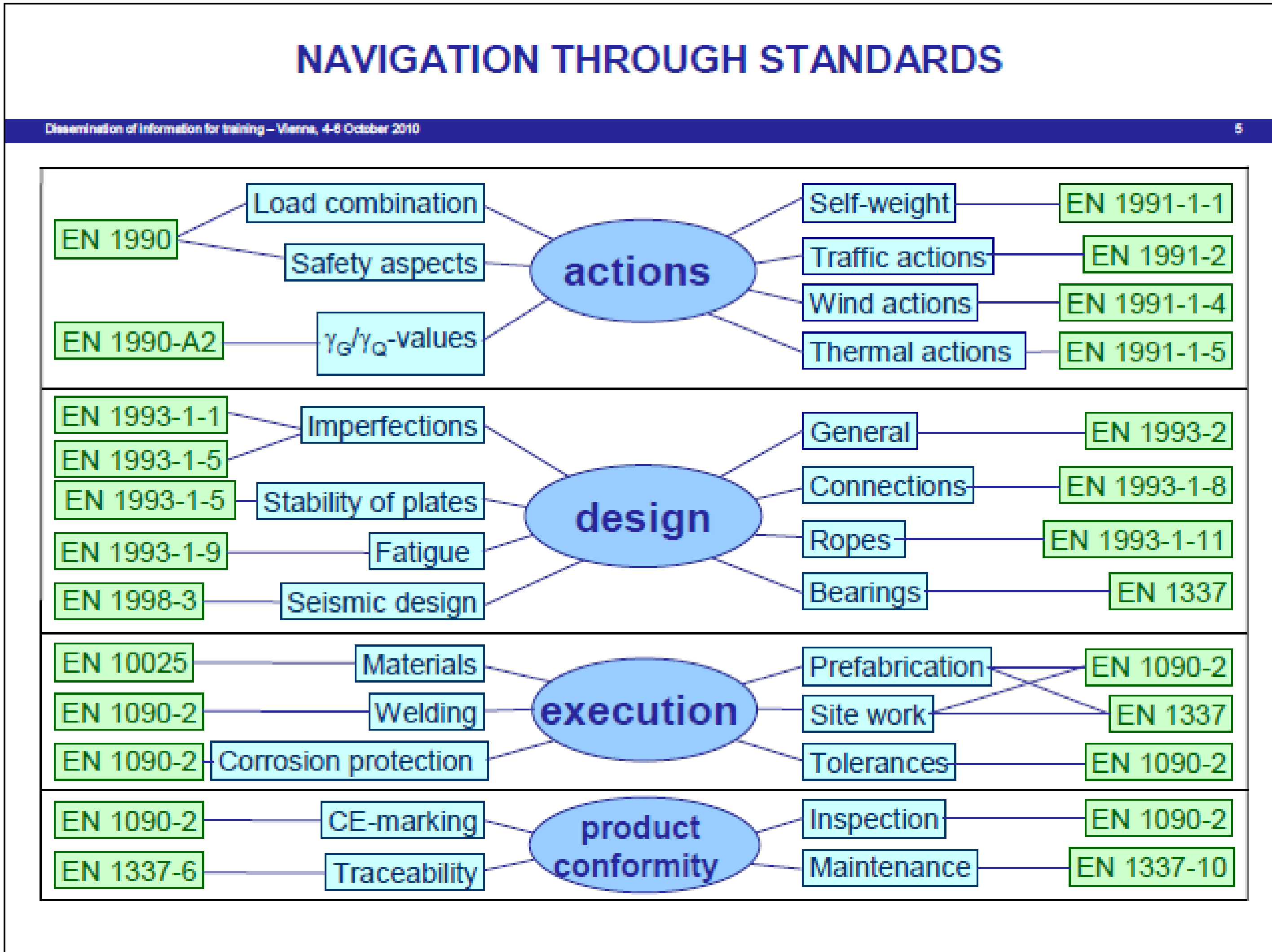
GENERAL

1. THE EUROPEAN STANDARD FAMILY AND STEEL BRIDGES



Tasks for designer and contractor

GENERAL


NAVIGATION THROUGH STANDARDS
Dissemination of information for training – Vienna, 4-6 October 2010
5
EN 1990
Load combination
Safety aspects
EN 1990-A2
γ_G/γ_Q-values
actions
Self-weight
EN 1991-1-1
Traffic actions
EN 1991-2
Wind actions
EN 1991-1-4
Thermal actions
EN 1991-1-5
EN 1993-1-1
EN 1993-1-5
Imperfections
EN 1993-1-5
Stability of plates
EN 1993-1-9
Fatigue
EN 1998-3
Seismic design
design
General
EN 1993-2
Connections
EN 1993-1-8
Ropes
EN 1993-1-11
Bearings
EN 1337
EN 10025
Materials
EN 1090-2
Welding
EN 1090-2
Corrosion protection
execution
Prefabrication
EN 1090-2
Site work
EN 1337
Tolerances
EN 1090-2
EN 1090-2
CE-marking
EN 1337-6
Traceability
product conformity
Inspection
EN 1090-2
Maintenance
EN 1337-10

GENERAL

Some of the main variables used in the design of steelwork are:

M	Bending moment	A	Area
N	Axial force	I	Second moment of area
V	Shear force	W	Section modulus
T	Torsional moment	$\bar{\lambda}$	Normalized slenderness
F	Force	χ	Reduction factor (for buckling)

Some of the main subscripts are

Ed	design effect	b	buckling
Rd	design resistance	cr	critical
Rk	characteristic resistance	op	out of plane
el	elastic	c	related to cross section
pl	plastic	y, z	related to y-y or z-z axis

Thus:

N_{Ed} is the design value of axial force (an effect)

$N_{c,Rd}$ is the design resistance of the cross section

$N_{b,Rd}$ is the design buckling resistance (of a member)

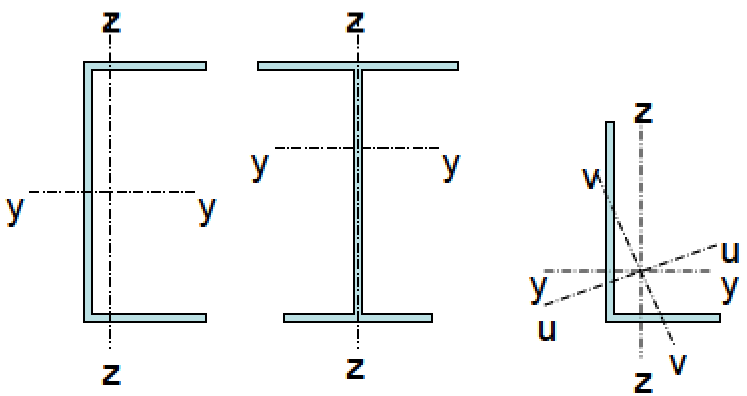
M_{cr} is the elastic critical moment (due to lateral torsion buckling of a beam)

GENERAL

Geometrical axes

The sign convention for axes differs from traditional UK convention but is compatible with many software analysis packages.

x – along the member



BASIS OF DESIGN

Ultimate limit state (ULS)

All beams should be designed to provide adequate strength to resist the design effects of the actions, using partial factors that are appropriate to the ULS. The following need to be considered.

- Material strength.
- Limitations on shape on account of local buckling of individual elements (i.e. webs and flanges).
- Moment resistance of cross sections.
- Effective sections (reductions for compression buckling and holes).
- Lateral torsional buckling.
- Web buckling (governed by depth to thickness ratio of web and panel size).
- Combined bending and shear effects.

Serviceability limit state (SLS)

Beams should also be designed to ensure that no yielding or permanent deformation takes place under the (lower) design effects that are appropriate to SLS.

BASIS OF DESIGN

Partial factors

The values of γ_M to be used are given in Table 2.1

Table 2.1 *Partial factors γ_M*

Partial factor on:		ULS	SLS
Resistance of cross sections	γ_{M0}	1.00	
Resistance of members to instability	γ_{M1}	1.10	
Resistance of joints	γ_{M2}	1.25	
Slip resistance (Category C joints)	γ_{M3}	1.25	
Slip resistance (Category B joints)	$\gamma_{M3,ser}$		1.10
Nominal stresses due to SLS characteristic loads	$\gamma_{M,ser}$		1.00

PROPERTIES OF MATERIALS

PROPERTIES OF MATERIALS

Yield strength

The yield strength of steel manufactured in accordance with BS EN 10025 should be taken as:

Grade S275	f_y	$= 275$ N/mm^2
Grade S355	f_y	$= 355$ N/mm^2

In most cases, grade S355 should be used, for economy.

Mechanical properties of steel

Modulus of elasticity,	E	$= 210\,000$ N/mm^2
Shear Modulus	G	$= 80\,000$ N/mm^2
Poisson's ratio	ν	$= 0.3$
Coefficient of thermal expansion	α	$= 12 \times 10^{-6}/°C$

CLASSIFICATION OF CROSS-SECTION

Cl 5.5, EC3-1-1

5.5 Classification of cross sections

5.5.1 Basis

(1) The role of cross section classification is to identify the extent to which the resistance and rotation capacity of cross sections is limited by its local buckling resistance.

5.5.2 Classification

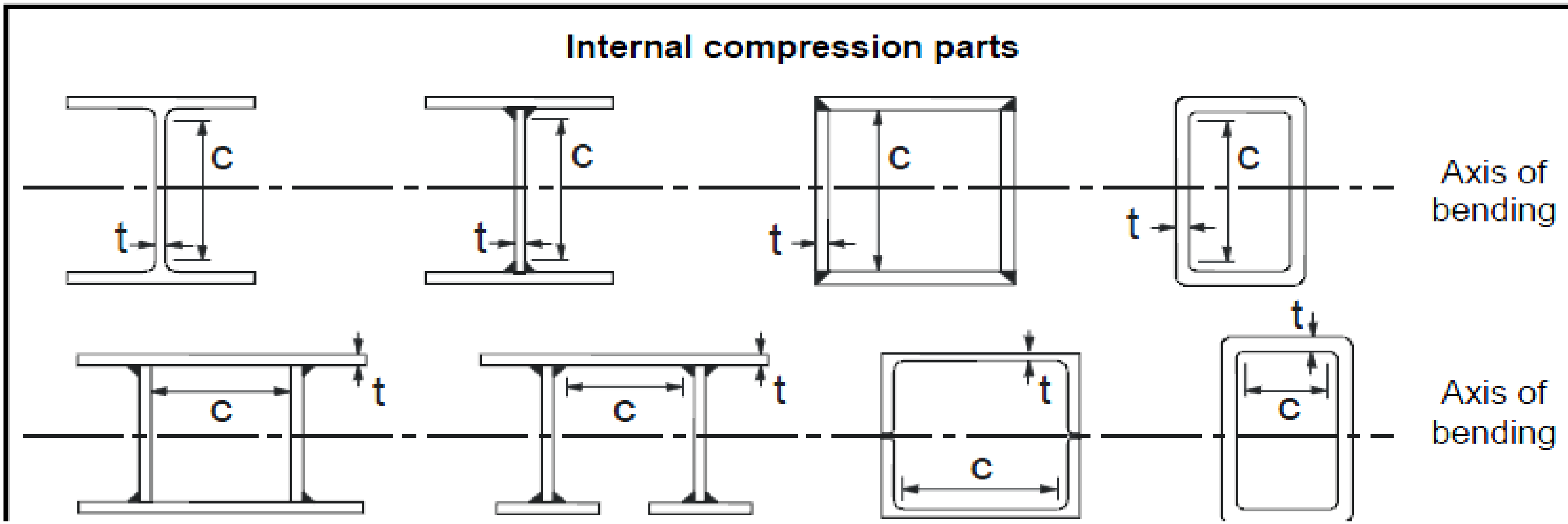
(1) Four classes of cross-sections are defined, as follows:

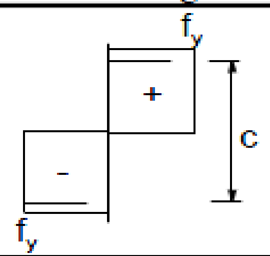
- Class 1 cross-sections are those which can form a plastic hinge with the rotation capacity required from plastic analysis without reduction of the resistance.
- Class 2 cross-sections are those which can develop their plastic moment resistance, but have limited rotation capacity because of local buckling.
- Class 3 cross-sections are those in which the stress in the extreme compression fibre of the steel member assuming an elastic distribution of stresses can reach the yield strength, but local buckling is liable to prevent development of the plastic moment resistance.
- Class 4 cross-sections are those in which local buckling will occur before the attainment of yield stress in one or more parts of the cross-section.

Web with neutral-axis at mid-depth and web subject to compression (rolled section)

BS EN 1993-1-1:2005
EN 1993-1-1:2005 (E)

Table 5.2 (sheet 1 of 3): Maximum width-to-thickness ratios for compression parts



<table>
<tr><th>Class</th><th>Part subject to bending</th><th>Part subject to compression</th><th>Part subject to bending and compression</th></tr>
<tr><td>Stress distribution in parts (compression positive)</td>
<td>
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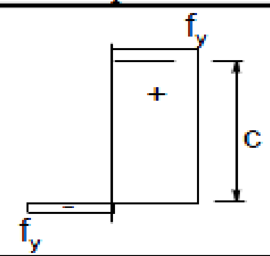
<td>
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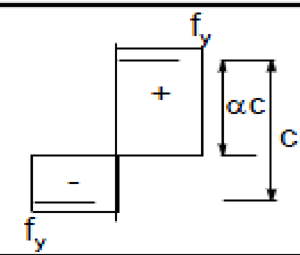
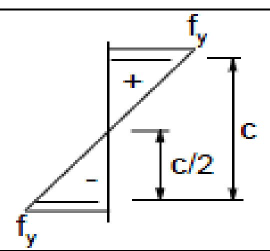
<td>
</td></tr>
<tr><td>1</td><td>$c/t \le 72\varepsilon$</td><td>$c/t \le 33\varepsilon$</td><td>when $\alpha > 0{,}5$: $c/t \le \dfrac{396\varepsilon}{13\alpha - 1}$
when $\alpha \le 0{,}5$: $c/t \le \dfrac{36\varepsilon}{\alpha}$</td></tr>
<tr><td>2</td><td>$c/t \le 83\varepsilon$</td><td>$c/t \le 38\varepsilon$</td><td>when $\alpha > 0{,}5$: $c/t \le \dfrac{456\varepsilon}{13\alpha - 1}$
when $\alpha \le 0{,}5$: $c/t \le \dfrac{41{,}5\varepsilon}{\alpha}$</td></tr>
<tr><td>Stress distribution in parts (compression positive)</td>
<td>
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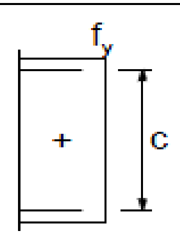
<td>
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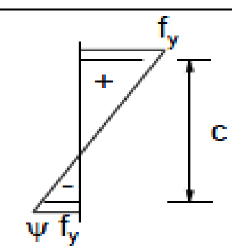
<td>
</td></tr>
<tr><td>3</td><td>$c/t \le 124\varepsilon$</td><td>$c/t \le 42\varepsilon$</td><td>when $\psi > -1$: $c/t \le \dfrac{42\varepsilon}{0{,}67 + 0{,}33\psi}$
when $\psi \le -1$*): $c/t \le 62\varepsilon(1-\psi)\sqrt{(-\psi)}$</td></tr>
</table>

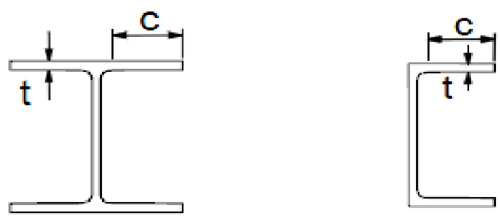
$\varepsilon = \sqrt{235/f_y}$	f_y	235	275	355	420	460
	ε	1,00	0,92	0,81	0,75	0,71

*) $\psi \le -1$ applies where either the compression stress $\sigma \le f_y$ or the tensile strain $\varepsilon_y > f_y/E$

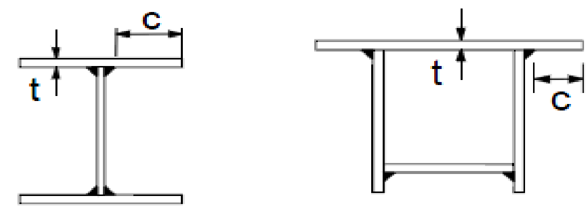
Outstand flange for rolled section

Table 5.2 (sheet 2 of 3): Maximum width-to-thickness ratios for compression parts

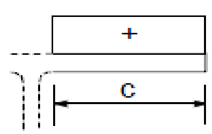
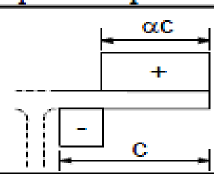
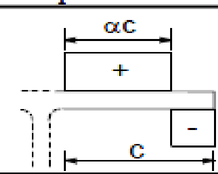
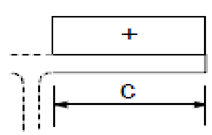
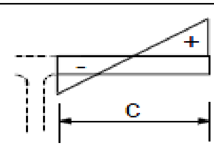
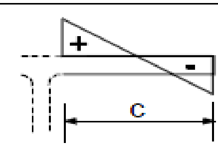
Outstand flanges



Rolled sections



Welded sections

Class	Part subject to compression	Part subject to bending and compression: Tip in compression	Part subject to bending and compression: Tip in tension
Stress distribution in parts (compression positive)			
1	$c/t \le 9\varepsilon$	$c/t \le \dfrac{9\varepsilon}{\alpha}$	$c/t \le \dfrac{9\varepsilon}{\alpha\sqrt{\alpha}}$
2	$c/t \le 10\varepsilon$	$c/t \le \dfrac{10\varepsilon}{\alpha}$	$c/t \le \dfrac{10\varepsilon}{\alpha\sqrt{\alpha}}$
Stress distribution in parts (compression positive)			
3	$c/t \le 14\varepsilon$	$c/t \le 21\varepsilon\sqrt{k_\sigma}$ For k_σ see EN 1993-1-5	

$\varepsilon = \sqrt{235/f_y}$	f_y	235	275	355	420	460
	ε	1,00	0,92	0,81	0,75	0,71

Summary of Table 5.2

Table 9.5 Maximum width-to-thickness ratios for compression elements (Table 5.2, EC3)

Type of element	*Class of element*		
	Class 1	*Class 2*	*Class 3*
Outstand flange for rolled section	$\frac{c}{t_f} \leq 9\varepsilon$	$\frac{c}{t_f} \leq 10\varepsilon$	$\frac{c}{t_f} \leq 14\varepsilon$
Web with neutral axis at mid depth, rolled section	$\frac{c^\star}{t_w} \leq 72\varepsilon$	$\frac{c^\star}{t_w} \leq 83\varepsilon$	$\frac{c^\star}{t_w} \leq 124\varepsilon$
Web subject to compression, rolled sections	$\frac{c^\star}{t_w} \leq 33\varepsilon$	$\frac{c^\star}{t_w} \leq 38\varepsilon$	$\frac{c^\star}{t_w} \leq 42\varepsilon$
f_y	235	275	355
ε	1	0.92	0.81

$c^\star = d$; $c = (b - t_w - 2r)/2$

RESISTANCE OF CROSS-SECTION

5.2 Design shear resistance according to EN 1993-1-5

EN 1993-1-5, §5.2

5.2.1 General

Design shear resistance is taken as

$$V_{b,Rd} = V_{bw,Rd} + V_{bf,Rd} \leq \frac{\eta f_{yw} h_w t}{\sqrt{3}\gamma_{M1}} \quad (5.9)$$

EN 1993-1-5, §5.2(1)

where

$$V_{bw,Rd} = \frac{\chi_w f_{yw} h_w t}{\sqrt{3}\gamma_{M1}} \text{ is the contribution from the web;} \quad (5.10)$$

$V_{bf,Rd}$ is the contribution from flanges;

χ_w is the reduction factor for the shear resistance of the sole web, depending on the web slenderness;

γ_{M1} is the partial factor for the resistance to instability;

η is the coefficient that includes the increase of shear resistance at smaller web slenderness;

NOTE 2 to 5.1 (2) of EN 1993-1-5 recommends the following values:

$\eta = 1,2$ for S235 to S460

$\eta = 1,0$ for steel grades over S460.

h_w, t are dimensions, see Figure 5.3.

RESISTANCE OF CROSS-SECTION

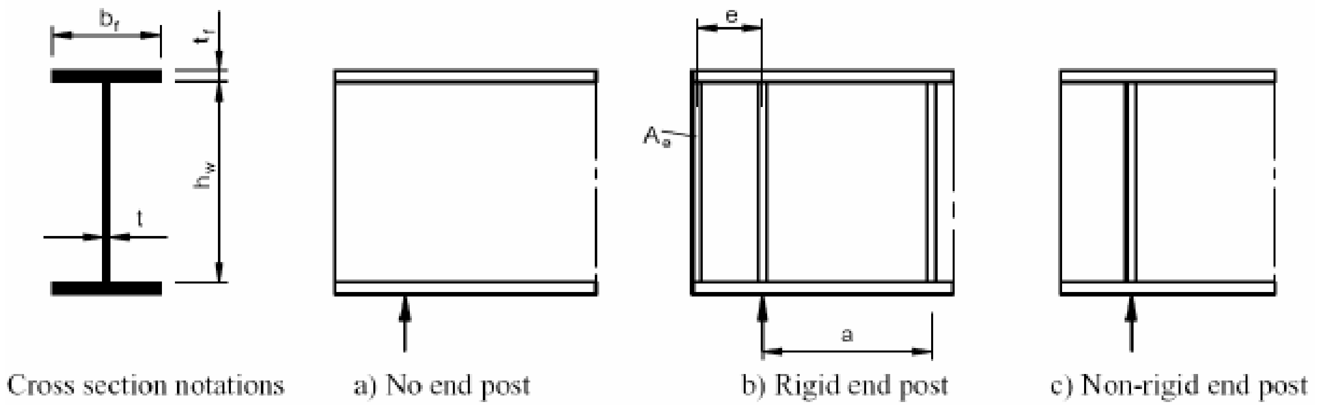


Figure 5.3: End-stiffeners

RESISTANCE OF CROSS-SECTION

Contribution from the web

The contribution from the web depends on its slenderness and thus on its tendency to buckle under shear loading. Usually in bridges transverse web stiffeners are provided at supports and at intermediate positions (intermediate transverse stiffeners are very effective in increasing the shear buckling resistance of a slender web by restricting the size of the rectangular panels within which buckling occurs - see further discussion below).

Web Design

Cl 5.1.(2), EC3-1-5, Note 2

For steel grades S460 and lower: η = 1.2
For steel grades higher than S460: η =1.0

Cl 5.2, EC3-1-5

Shear buckling resistance,

$$\boxed{V_{b,Rd} = V_{bw,Rd} + V_{bf,Rd} \leq \frac{\eta f_{yw} h_w t}{\sqrt{3}\gamma_{M1}}}$$

Web Design

Cl 5.1.(2), EC3-1-5

For unstiffened webs : $\boxed{\dfrac{h_w}{t} < \dfrac{72}{\eta}\varepsilon}$

For stiffened webs : $\boxed{\dfrac{h_w}{t} < \dfrac{31}{\eta}\varepsilon\sqrt{k_\tau}}$

if these limits are exceeded resistance to shear buckling should be checked and transverse stiffeners should be provided at the supports

where

i) $\dfrac{a}{h_w} \geq 1.0, \quad k_\tau = 5.34 + 4.00\left(\dfrac{h_w}{a}\right)^2$

ii) $\dfrac{a}{h_w} < 1.0, \quad k_\tau = 4.00 + 5.34\left(\dfrac{h_w}{a}\right)^2$

Web Design

where the contribution from the web is given by,

$$\boxed{V_{bw,Rd} = \frac{\chi_w f_{yw} h_w t}{\sqrt{3}\gamma_{M1}}}$$

and the contribution from the flange is given by (Cl 5.4, EC3-1-5),

$$\boxed{V_{bf,Rd} = \frac{b_f\, t^2{}_f\, f_{yf}}{c\, \gamma_{M1}}\left(1 - \left(\frac{M_{Ed}}{M_{f,Rd}}\right)^2\right)}$$

Contribution from the Web

Cl 5.3, EC3-1-5

(1) For webs with transverse stiffeners at supports only and for webs with either intermediate transverse stiffeners or longitudinal stiffeners or both, the factor χ_w for the contribution of the web to the shear buckling resistance should be obtained from Table 5.1 or Figure 5.2.

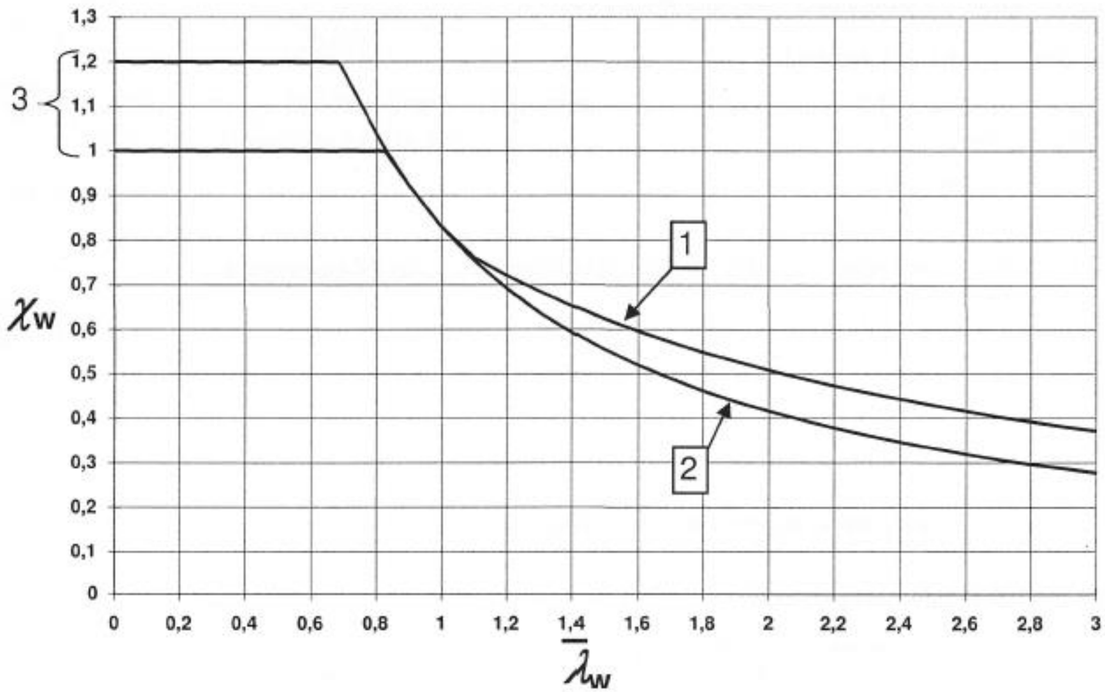
Table 5.1: Contribution from the web χ_w to shear buckling resistance

	Rigid end post	Non-rigid end post
$\bar{\lambda}_w < 0{,}83/\eta$	η	η
$0{,}83/\eta \leq \bar{\lambda}_w < 1{,}08$	$0{,}83/\bar{\lambda}_w$	$0{,}83/\bar{\lambda}_w$
$\bar{\lambda}_w \geq 1{,}08$	$1{,}37/(0{,}7+\bar{\lambda}_w)$	$0{,}83/\bar{\lambda}_w$

NOTE: See 6.2.6 in EN 1993-1-1.

Contribution from the Web

Cl 5.3, EC3-1-5



1 Rigid end post
2 Non-rigid end post
3 Range of recommended η

Figure 5.2: Shear buckling factor χ_w

Contribution from the Web

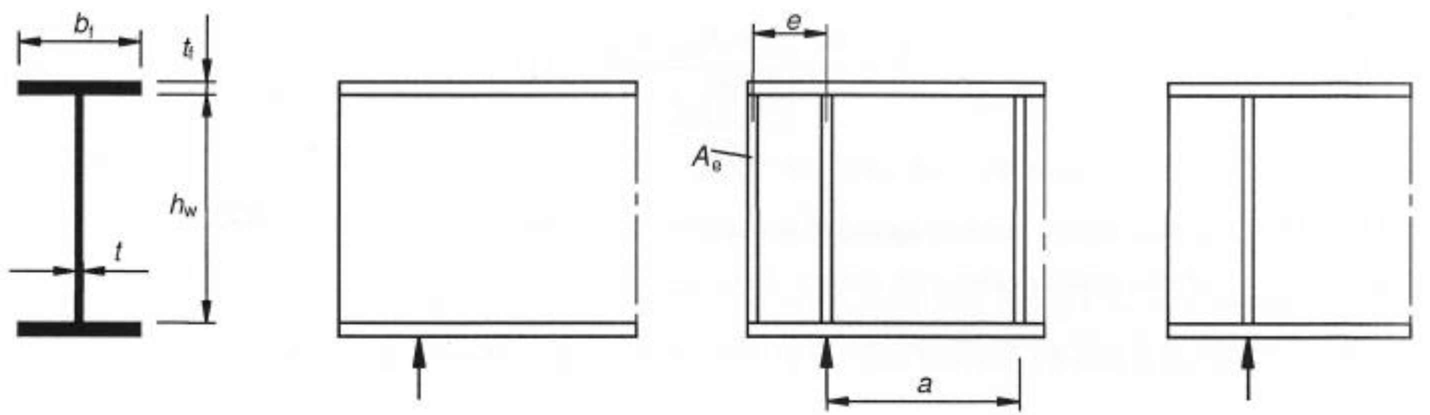
Cl 5.3, EC3-1-5

(2) Figure 5.1 shows various end supports for girders:

a) No end post, see 6.1 (2), type c);

b) Rigid end posts, see 9.3.1; this case is also applicable for panels at an intermediate support of a continuous girder;

c) Non rigid end posts see 9.3.2.



Cross section notations   a) No end post   b) Rigid end post   c) Non-rigid end post

Figure 5.1: End supports

Contribution from the Web

Cl 5.3, EC3-1-5

(3) The slenderness parameter $\bar{\lambda}_w$ in Table 5.1 and Figure 5.2 should be taken as:

$$\bar{\lambda}_w = 0,76\sqrt{\frac{f_{yw}}{\tau_{cr}}} \tag{5.3}$$

where $\tau_{cr} = k_\tau\ \sigma_E$ (5.4)

NOTE 1: Values for σ_E and k_τ may be taken from Annex A.

NOTE 2: The slenderness parameter $\bar{\lambda}_w$ may be taken as follows:

a) transverse stiffeners at supports only:

$$\bar{\lambda}_w = \frac{h_w}{86,4\ t\ \varepsilon} \tag{5.5}$$

b) transverse stiffeners at supports and intermediate transverse or longitudinal stiffeners or both:

$$\bar{\lambda}_w = \frac{h_w}{37,4\ t\ \varepsilon\ \sqrt{k_\tau}} \tag{5.6}$$

Contribution from the Web

Cl 5.3, EC3-1-5

(4) The second moment of area of a longitudinal stiffener should be reduced to 1/3 of its actual value when calculating k_τ. Formulae for k_τ taking this reduction into account in A.3 may be used.

(5) For webs with longitudinal stiffeners the slenderness parameter $\bar{\lambda}_w$ in (3) should not be taken as less than

$$\bar{\lambda}_w = \frac{h_{wi}}{37{,}4\, t\, \varepsilon \sqrt{k_{\tau i}}} \quad (5.7)$$

where h_{wi} and $k_{\tau i}$ refer to the subpanel with the largest slenderness parameter $\bar{\lambda}_w$ of all subpanels within the web panel under consideration.

Contribution from Flanges

Cl 5.4, EC3-1-5

(1) When the flange resistance is not completely utilized in resisting the bending moment ($M_{Ed} < M_{f,Rd}$) the contribution from the flanges should be obtained as follows:

$$V_{bf,Rd} = \frac{b_f\, t_f^2\, f_{yf}}{c\, \gamma_{M1}}\left(1-\left(\frac{M_{Ed}}{M_{f,Rd}}\right)^2\right) \tag{5.8}$$

b_f and t_f are taken for the flange which provides the least axial resistance,

b_f being taken as not larger than $15\varepsilon t_f$ on each side of the web,

$M_{f,Rd} = \frac{M_{f,k}}{\gamma_{M0}}$ is the moment of resistance of the cross section consisting of the effective area of the flanges only,

$$c = a\left(0{,}25 + \frac{1{,}6\, b_f t_f^2\, f_{yf}}{t\, h_w^2\, f_{yw}}\right) = a\left(0.25 + 1.6\,\frac{M_{pl,f}}{M_{pl,w}}\right)$$

Contribution from Flanges

Cl 5.4, EC3-1-5

(2) When an axial force N_{Ed} is present, the value of $M_{f,Rd}$ should be reduced by multiplying it by the following factor:

$$\left(1-\frac{N_{Ed}}{\dfrac{(A_{f1}+A_{f2})f_{yf}}{\gamma_{M0}}}\right) \tag{5.9}$$

where A_{f1} and A_{f2} are the areas of the top and bottom flanges respectively.

5.5 Verification

(1) The verification should be performed as follows:

$$\eta_3=\frac{V_{Ed}}{V_{b,Rd}}\leq 1{,}0 \tag{5.10}$$

where V_{Ed} is the design shear force including shear from torque.

GUIDED TUTORIAL 1 (1 hour)

Continue…

Proposed a suitable size of plate girder in grade S355 steel to carry a characteristic variable actions of 150 kN/m over a span of 20m. The compression flange is fully restrained against Lateral Torsional Buckling (LTB).

Design of Stiffeners

Cl 9.3.1, EC3-1-5

Rigid End Post

This may either be a set of flats welded at the end and above the support with the centroids a distance e apart, or the end post may comprise a rolled section in which case e is the distance between the flange centroids.

Assuming the end post is simply supported, the maximum moment $M_{\max}$ is given by

$$M_{\max} = \frac{q_{\mathrm{h}} h_{\mathrm{w}}^2}{8} \tag{5.160}$$

Design of Stiffeners

Cl 9.3.1, EC3-1-5

Rigid End Post

The section modulus, ignoring any contribution from the web, W is given by $A_{\min}e$, thus assuming the maximum stress is given by f_y, then

$$\sigma_{\max} = f_y = \frac{M_{\max}}{W} = \frac{\dfrac{32t^2 f_y}{h_w}\dfrac{h_w^2}{8}}{A_{\min}e} \qquad (5.161)$$

Thus the minimum cross-sectional area of each pair $A_{\min}$ is given by

$$A_{\min} = \frac{4h_w^2 t}{e} \qquad (5.162)$$

with $e > 0{,}1h_w$.

Design of Stiffeners

Cl 9.3.1, EC3-1-5

Rigid End Post

The end panel may be designed as non-rigid shear panel carrying the whole of the applied shear. the required value of χ_w is given by

$$\chi_w = \frac{V_{Ed}\sqrt{3}\gamma_{M1}}{f_{yw}h_w t} \tag{5.163}$$

the normalized web slenderness $\bar{\lambda}_w$ is given by

$$\bar{\lambda}_w = \frac{0,83}{\chi_w} \tag{5.164}$$

Design of Stiffeners

Cl 9.3.1, EC3-1-5

Rigid End Post

The buckling parameter k_τ is given as

$$k_\tau = \left(\frac{h_w}{37{,}4 t \varepsilon \bar{\lambda}_w}\right)^2 \qquad (5.165)$$

As $a/h_w < 1$, the required panel width a is given as

$$a = h_w \sqrt{\frac{5{,}35}{k_\tau - 4{,}00}} \qquad (5.166)$$

Design of Stiffeners

Cl 9.2.1, EC3-1-5

Transverse Stiffeners

In the absence of transverse loads or axial forces in the stiffeners, then the strength and deflection criteria are satisfied if they have a second moment of area given by

$$I_{st} = \frac{\sigma_M}{E}\left(\frac{b}{\pi}\right)^4\left(1 + w_0\frac{300}{b}u\right) \tag{5.167}$$

where σ_M is given by

$$\sigma_M = \frac{\sigma_{cr,c}}{\sigma_{cr,p}}\frac{N_{Ed}}{b}\left(\frac{1}{a_1} + \frac{1}{a_2}\right) \tag{5.168}$$

where a_1 and a_2 are the panel lengths either side of the stiffener under consideration, N_{Ed} is the larger compressive force in the adjacent panels, b is the height of the stiffener.

Design of Stiffeners

Cl 9.2.1, EC3-1-5

Transverse Stiffeners

The initial imperfection w_0 is given as

$$w_0 = \frac{1}{300}\text{LEAST}(a_1, a_2, b) \tag{5.169}$$

The parameter u is given by

$$u = \frac{\pi^2 E e_{\max}}{\frac{300 b f_y}{\gamma_{M1}}} \geq 1{,}0 \tag{5.170}$$

The distance $e_{\max}$ is taken from the extreme fibre of the stiffener to the centroid of the stiffener.

Design of Stiffeners

Cl 9.2.1, EC3-1-5

Transverse Stiffeners

The critical stress for plate between vertical stiffeners $\sigma_{cr,c}$ is given by

$$\sigma_{cr,c} = \frac{\pi^2 E t^2}{12(1-\nu^2)a^2} \tag{5.171}$$

To avoid lateral torsional buckling of the stiffener,

$$\frac{I_T}{I_p} \geq 5{,}3\frac{f_y}{E} \tag{5.172}$$

where I_T and I_p are St. Venant torsional constant for the stiffener alone and I_p is the polar second moment of area about the edge fixed to the plate.

Design of Stiffeners

Cl 9.3.3, EC3-1-5

Intermediate Transverse Stiffeners

The force $N_{s,Rd}$ to be resisted by a stiffener is given by

$$N_{s,Rd} = V_{Ed} - \chi_w h_w t \frac{\frac{f_{yw}}{\sqrt{3}}}{\gamma_{M1}} \qquad (5.176)$$

Note, χ_w is calculated for the web panel between adjacent stiffeners assuming the stiffener under consideration is removed. In the case of variable shear, then the check is performed at a distance $0{,}5h_w$ from the edge of the pane with the larger shear force.

To determine the buckling resistance of the stiffener a portion of the web may taken into account . A section of the web in length equal to $15\varepsilon t$ either side of the stiffener may be considered (cl 9.1, EN 1933-1-3) (Fig. 5.23)

Design of Stiffeners

Cl 9.3.3, EC3-1-5

Intermediate Transverse Stiffeners

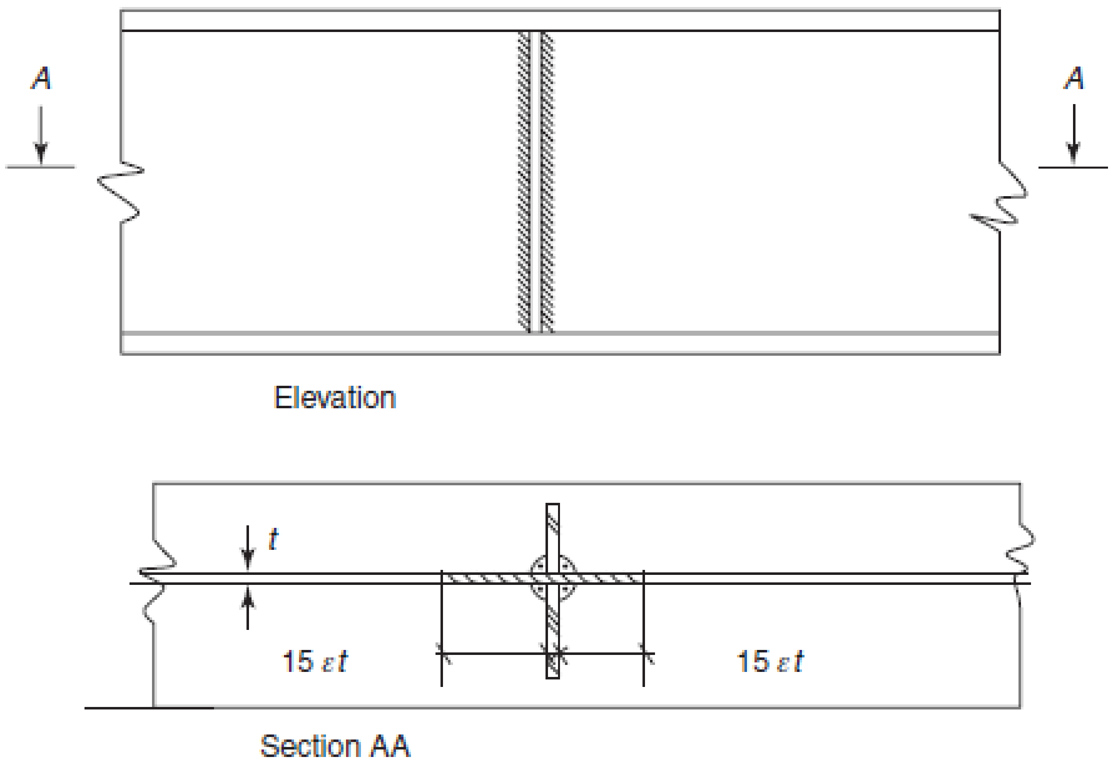


FIGURE 5.23 Stiffener geometry

Design of Stiffeners

Cl 9.3.3, EC3-1-5

Intermediate Transverse Stiffeners

For a symmetric stiffener, the effective area A_e is given by

$$A_{equiv} = A_{st} + 30\varepsilon t^2 \tag{5.177}$$

and the effective second moment of area I_{equiv} by

$$I_{equiv} = I_{st} + \frac{1}{12} 30\varepsilon t^4 \tag{5.178}$$

where A_{st} is the area of the stiffener and I_{st} is the second moment of area of the stiffener. For end stiffeners the co-efficient of 30 in Eqs (5.177) and (5.178) should be replaced by 15.

The effective length of the stiffener may be taken as $0,75h_w$ and buckling curve 'c' used to determine the strength reduction factor (cl 9.4 EN 1993-1-5).

Design of Stiffeners

Cl 9.3.3, EC3-1-5

Intermediate Transverse Stiffeners

The minimum second moment of area I_s is given by

for $a/h_w < \sqrt{2}$

$$I_{st} \geq 1{,}5\frac{h_w^3 t^3}{a^2} \qquad (5.179)$$

for $a/h_w \geq \sqrt{2}$

$$I_{st} \geq 0{,}75 h_w t^3 \qquad (5.180)$$

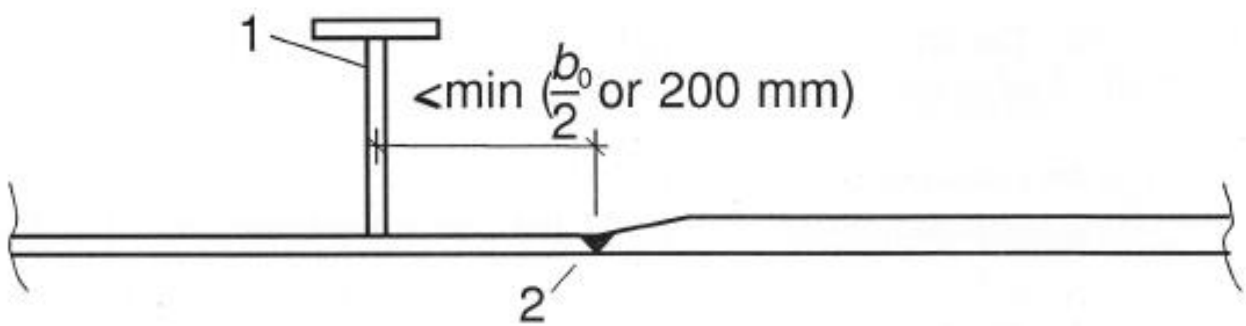
It can be demonstrated that for compression buckling a change from 1 to 2 half sine waves occurs at $a/h_w = \sqrt{2}$, and that thereafter the buckling co-efficient is sensibly independent of the aspect ratio of the panel. Thus Eq. (5.180) is determined from Eq. (5.179) by substituting $a = h_w\sqrt{2}$ (cl 9.3.3 (3) EN 1993-1-5).

Design of Stiffeners

Cl 9.2.3, EC3-1-5

Plate Splices

The splice whether in the web or flanges, should ideally occur at a transverse stiffener. If not then the stiffener should be at a distance no greater than $b_0/2$ along the thinner plate where b_0 is the depth of the web (or the least spacing of longitudinal stiffeners).



1 Transverse stiffener
2 Transverse weld

Design of Stiffeners

Cl 9.3.5, EC3-1-5

Longitudinal Welds

The weld between the web and flange(s) should be designed for a shear flow of $V_{\mathrm{Ed}}/h_{\mathrm{w}}$, provided

$$V_{\mathrm{Ed}} < \chi_{\mathrm{w}} h_{\mathrm{w}} t \frac{\frac{f_{\mathrm{yw}}}{\sqrt{3}}}{\gamma_{\mathrm{M1}}} \tag{5.181}$$

If the condition in Eq. (5.181) is not satisfied, the welds should be designed under a shear flow of $\eta t (f_{\mathrm{yw}}/\sqrt{3})/\gamma_{\mathrm{M1}}$.

GUIDED TUTORIAL 1 (1 hour)

Continue….

Proposed a suitable size of plate girder in grade S355 steel to carry a characteristic variable actions of 150 kN/m over a span of 20m. The compression flange is fully restrained against Lateral Torsional Buckling (LTB).


LUNCH
BREAK

LECTURE 4:
MAIN GIRDER SPLICES

- Introduction
- Slip resistance of bolt
- Bolt spacing and end edge distance
- Splice configuration

LECTURE 4: MAIN GIRDER SPLICES

BOLTED CONNECTIONS

General

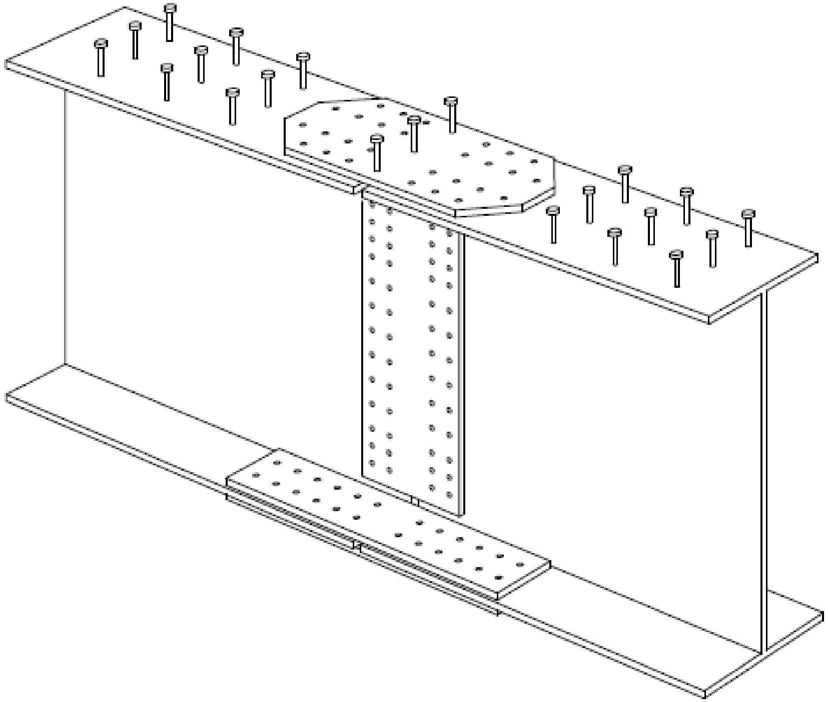
Bolted connections in bridges are made using preloaded bolts (commonly referred to as high strength friction grip bolts). These bolts are tightened in such a way that a reliable preload is achieved in the bolt. This preload allows shear to be transferred between the interfaces (faying surfaces) by friction – i.e. the surfaces do not slip, one relative to another, until the frictional resistance has been overcome.

Such bolts are used to ensure that there is no movement (slip) at a bolted joint under normal (i.e. serviceability limit state) conditions. In most connections it is acceptable to allow the bolts to slip at the higher ULS loads, at which time they act in bearing and shear (this is usually significantly greater than the ULS friction capacity).

LECTURE 4: MAIN GIRDER SPLICES

Girder splices

Girders are usually connected using 'double cover plates'. See Figure 14.1. These are additional plates, one on each side of the element to be spliced. The bolts are then in 'double shear'. Bracing is usually connected simply by lapping, in which case the bolts are in single shear.



LECTURE 4: MAIN GIRDER SPLICES

Cross girder connections in ladder decks

The intermediate cross girders in ladder deck bridges are usually connected to the main girders using a simple lapped connection of the web of the cross girder onto a transverse web stiffener. This detail is illustrated in Figure 14.2. Cross girders at supports are usually connected using double web cover plates (as in a girder splice) but that connection detail would not usually need to be considered in a simplified design.

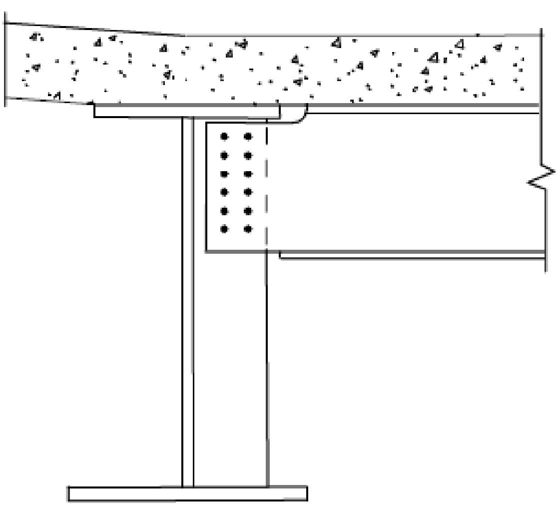


Figure 14.2 *Connection of intermediate cross girder in a ladder deck bridge*

LECTURE 4: MAIN GIRDER SPLICES

Slip resistant connections

The design slip resistance of a preloaded bolted connection (HSFG bolted joint) is given by:

$$F_{s,Rd} = \frac{k_s n \mu F_{p,c}}{\gamma_{M3}} \qquad (14.1)$$

where

$F_{p,c}$ is the preload in the bolt

n is the number of friction surfaces

μ is the slip factor

k_s is a factor related to the type of hole (normal, oversize, etc.)

Typical friction values at SLS, for M24 and M27 bolts grade 8.8 in normal clearance holes, and for grit blasted steel surfaces, are given in Table 14.1.

LECTURE 4:
MAIN GIRDER SPLICES

Table 14.1 *Slip resistance of preloaded connections*

	SLS capacity (KN)		ULS capacity (KN)	
	Double shear	Single shear	Double shear	Single shear
M24	180	90	158	79
M27	234	117	206	103

LECTURE 4: MAIN GIRDER SPLICES

Shear resistance of bolted connection

Where a bolt does slip into bearing and shear, its shear resistance is given by:

$$F_{v,Rd} = \frac{\alpha_v f_{ub} A}{\gamma_{M2}} \text{ per shear plane} \tag{14.2}$$

Where

f_{ub} is the ultimate tensile strength of the bolt

A is the tensile area of the bolt

α_v = 0.6 for grade 8.8 bolts and = 0.5 for grade 10.9 bolts

Typical shear capacities, for M24 and M27 bolts, grade 8.8, in normal clearance holes, are given in Table 14.2.

Table 14.2 *Shear values of HSFG bolted connections*

	ULS capacity (kN)	
	Double shear	Single shear
M24	271	136
M27	353	176

The above values apply where the shear planes pass through the unthreaded portion of the bolt, which is the usual design condition.

LECTURE 4:
MAIN GIRDER SPLICES

Cover plates

For simplified design, cover plates should be provided on both faces of the part being connected. No specific rules for plate thickness are given in EN 1993-1-8 but in normal practice the cover plates should be at least half as thick as the element joined and not less than 8 mm thick.

Spacing of bolts

There are limitations on the maximum and minimum spacing of bolts. The maximum spacing depends on the thickness of the outer cover plates. The minimum spacing depends on the diameter of the bolt. For bolts arranged in an orthogonal pattern (lines of bolts parallel to the edges of the connected parts) a simplified table of maximum spacing is given in Table 14.3

LECTURE 4: MAIN GIRDER SPLICES

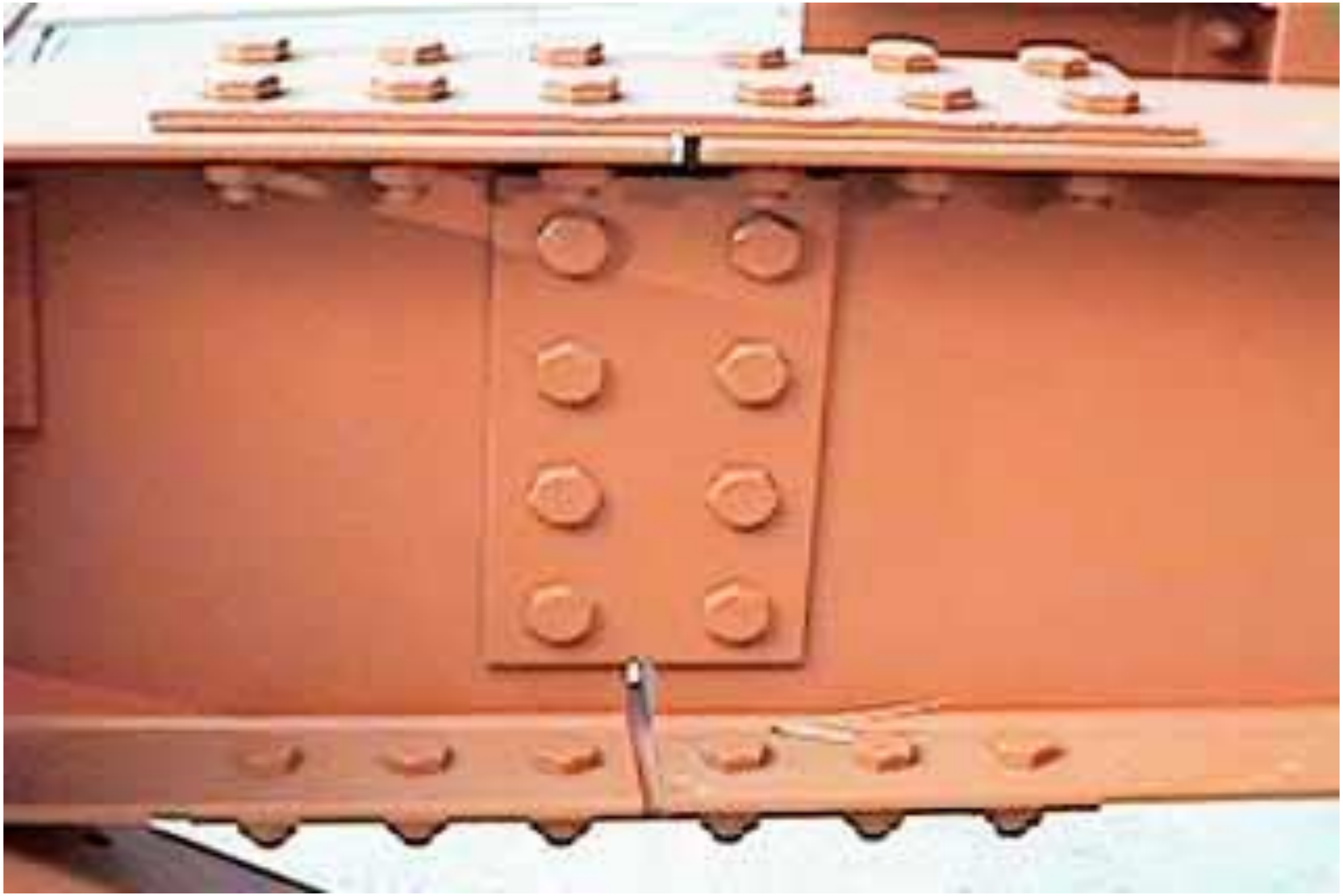
Table 14.3 *Maximum spacing of bolts*

Cover plate (mm)	Maximum spacing (mm)
8	112
10	140
12	168
15 and over	200*

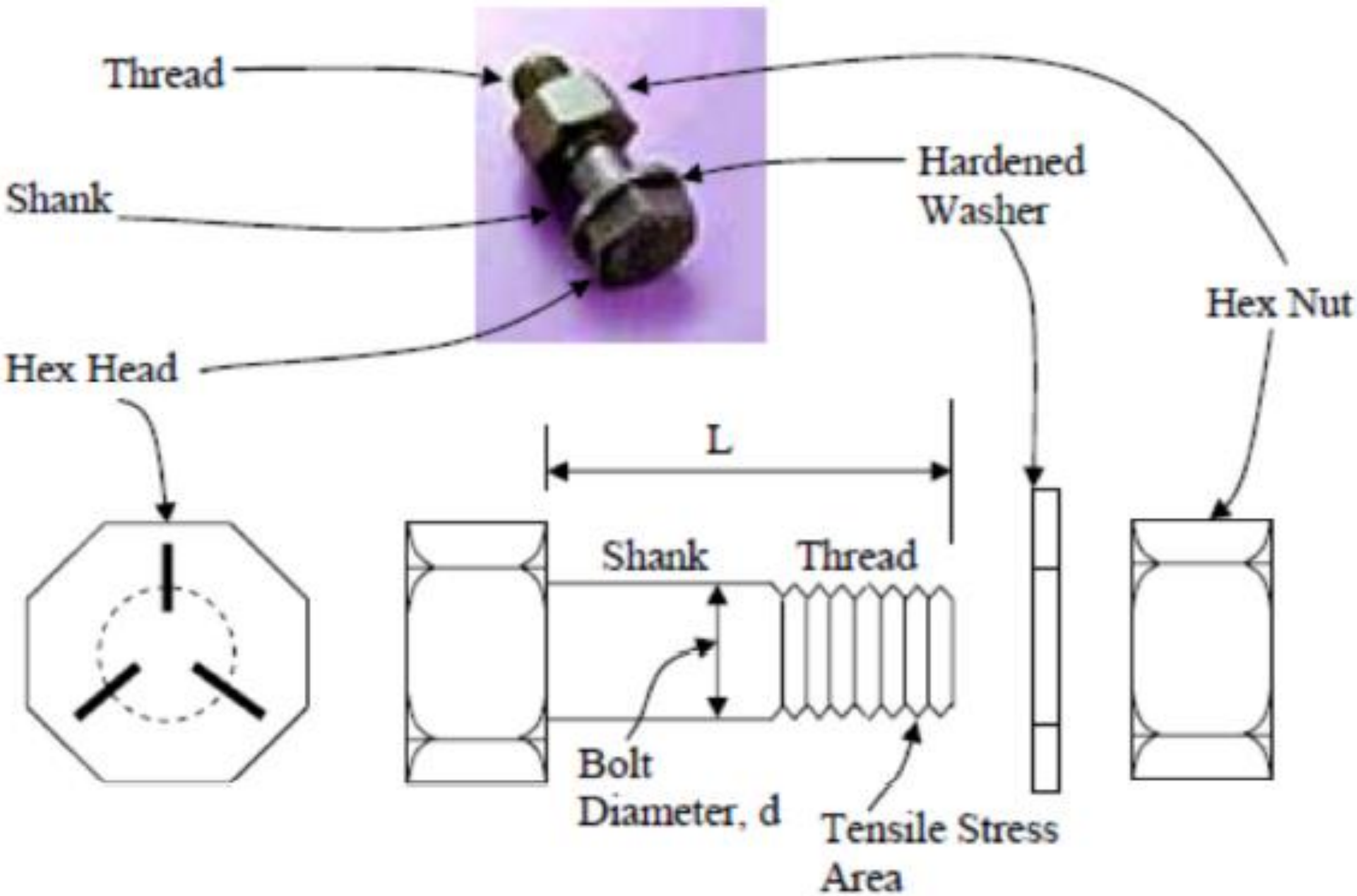
* For weathering steel the limit is 175 mm

The minimum spacing is 57 mm for M24 bolts and 66 mm for M27 bolts (use rounded minimum dimensions of 60 mm and 70 mm respectively).

BOLTED CONNECTIONS



1. A typical bolt used in bolting connections is shown below.



2. All bolts, nuts and washers should comply with the standards listed in Cl 3.1.1(1) and Group 4 Reference Standards in Cl 1.2.4 of EC3-1-8.

3. EC3 provides for the use of 'ordinary bolts' (non-preloaded bolts) or 'high strength bolts' (preloaded bolts) in classes from 4.6 up to 10.9. Generally class 8.8 M20 bolts should be used for connections in members which will accommodate this size, and M16 used for smaller members. Heavily loaded connections may require M24 or M30 bolts, or the use of class 10.9 bolts. As far as possible only one size and grade should be used on a project.

4. The yield strength f_{yb} and the ultimate tensile strength f_{ub} for bolt classes are given in Table 3.1, Cl 3.1.1(3) of EC3-1-8.

Table 3.1: Nominal values of the yield strength f_{yb} and the ultimate tensile strength f_{ub} for bolts

Bolt class	4.6	4.8	5.6	5.8	6.8	8.8	10.9
f_{yb} (N/mm^2)	240	320	300	400	480	640	900
f_{ub} (N/mm^2)	400	400	500	500	600	800	1000

5. The connection should be designed on a realistic, and consistent, assumption of the distribution of internal forces in the connection, which are in equilibrium with the externally applied loads. Each element in the connection must have sufficient resistance and deformation capacity.

6. The advantages of using bolted joints are fast constructions at site, require less supervision than welding, supports load as soon as the bolts are tightened and accommodates minor discrepancies in dimensions.

7. The disadvantages of bolted connections are that for large forces the space required for the joints is extensive, and the connection is not as rigid as a welded connection even when preloaded bolts are used.

Categories of Bolted Connections

(EC3-1-8, Cl 3.4)

1. Bolted connections loaded in shear should be designed as one of the following:

- Category A: Bearing type
- Category B: Slip-resistant at serviceability limit state
- Category C: Slip-resistant at ultimate limit state

*Note that for Category C connections where the connected elements are in tension, the design plastic resistance of the net cross-section at bolt holes should also be verified.

2. Bolted connections loaded in tension should be designed as one of the following:

   – Category D: Non-preloaded
   – Category E: Preloaded

3. The design checks for these connections are summarized in Table 3.2.

4. For wind and stability bracings, bolts in Category A connections may be used.

5. Category D connections may be used in connections designed to resist normal wind loads.

Table 3.2: Categories of bolted connections

Category	Criteria	Remarks
	Shear connections	
A bearing type	$F_{v,Ed} \leq F_{v,Rd}$ $F_{v,Ed} \leq F_{b,Rd}$	No preloading required. Bolt classes from 4.6 to 10.9 may be used.
B slip-resistant at serviceability	$F_{v,Ed,ser} \leq F_{s,Rd,ser}$ $F_{v,Ed} \leq F_{v,Rd}$ $F_{v,Ed} \leq F_{b,Rd}$	Preloaded 8.8 or 10.9 bolts should be used. For slip resistance at serviceability see 3.9.
C slip-resistant at ultimate	$F_{v,Ed} \leq F_{s,Rd}$ $F_{v,Ed} \leq F_{b,Rd}$ AC2 $\sum F_{v,Ed} \leq N_{net,Rd}$ AC2	Preloaded 8.8 or 10.9 bolts should be used. For slip resistance at ultimate see 3.9. $N_{net,Rd}$ see 3.4.1(1) c).
	Tension connections	
D non-preloaded	$F_{t,Ed} \leq F_{t,Rd}$ $F_{t,Ed} \leq B_{p,Rd}$	No preloading required. Bolt classes from 4.6 to 10.9 may be used. $B_{p,Rd}$ see Table 3.4.
E preloaded	$F_{t,Ed} \leq F_{t,Rd}$ $F_{t,Ed} \leq B_{p,Rd}$	Preloaded 8.8 or 10.9 bolts should be used. $B_{p,Rd}$ see Table 3.4.
The design tensile force $F_{t,Ed}$ should include any force due to prying action, see 3.11. Bolts subjected to both shear force and tensile force should also satisfy the criteria given in Table 3.4.		

Spacing, End and Edge Distances

(EC3-1-8, Cl 3.5)

1. A summary is given in Table 3.3 of EC3-1-8. A minimum spacing in the direction of the load, p_1, is specified to prevent excessive reduction in a cross-sectional area of a member, to provide sufficient space to tighten the bolts and to prevent overlapping of the washers.

2. Other critical spacing distances are specified to prevent buckling of plates in compression between bolts, to ensure that bolts act together as a group to resist forces and to minimise corrosion.

3. End and edge distances are specified to resist the load, to prevent local buckling, to limit corrosion and to provide space for the bolt head, washer and nut.

Table 3.3: Minimum and maximum spacing, end and edge distances

Distances and spacings, see Figure 3.1	Minimum	Maximum [1) 2) 3)]		
		Structures made from steels conforming to EN 10025 except steels conforming to EN 10025-5		Structures made from steels conforming to EN 10025-5
		Steel exposed to the weather or other corrosive influences	Steel not exposed to the weather or other corrosive influences	Steel used unprotected
End distance e_1	$1{,}2d_0$	$4t + 40$ mm		The larger of $8t$ or 125 mm
Edge distance e_2	$1{,}2d_0$	$4t + 40$ mm		The larger of $8t$ or 125 mm
Distance e_3 in slotted holes	$1{,}5d_0$ [4)]			
Distance e_4 in slotted holes	$1{,}5d_0$ [4)]			
Spacing p_1	$2{,}2d_0$	The smaller of $14t$ or 200 mm	The smaller of $14t$ or 200 mm	The smaller of $14t_{min}$ or 175 mm
Spacing $p_{1,0}$		The smaller of $14t$ or 200 mm		
Spacing $p_{1,i}$		The smaller of $28t$ or 400 mm		
Spacing p_2 [5)]	$2{,}4d_0$	The smaller of $14t$ or 200 mm	The smaller of $14t$ or 200 mm	The smaller of $14t_{min}$ or 175 mm

1) Maximum values for spacings, edge and end distances are unlimited, except in the following cases:

- for compression members in order to avoid local buckling and to prevent corrosion in AC2 exposed members (the limiting values are given in the table) and; AC2
- for exposed tension members AC2 to prevent corrosion (the limiting values are given in the table). AC2

2) The local buckling resistance of the plate in compression between the fasteners should be calculated according to EN 1993-1-1 using $0,6\,p_1$ as buckling length. Local buckling between the fasteners need not to be checked if p_1/t is smaller than $9\,\varepsilon$. The edge distance should not exceed the local buckling requirements for an outstand element in the compression members, see EN 1993-1-1. The end distance is not affected by this requirement.

3) t is the thickness of the thinner outer connected part.

4) The dimensional limits for slotted holes are given in 1.2.7 Reference Standards: Group 7.

5) For staggered rows of fasteners a minimum line spacing of $p_2 = 1{,}2d_0$ may be used, provided that the minimum distance, L, between any two fasteners is greater or equal than $2{,}4d_0$, see Figure 3.1b).

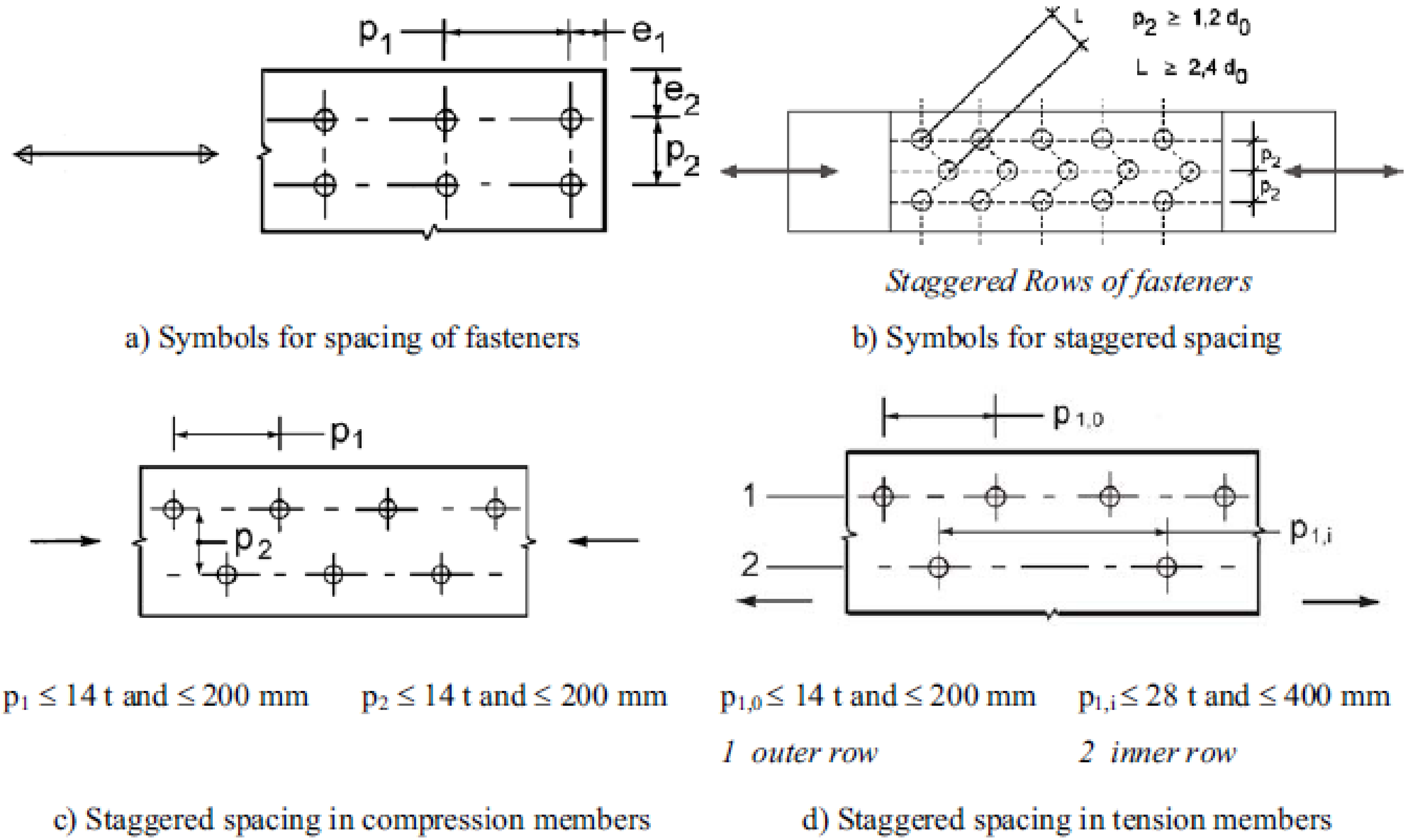
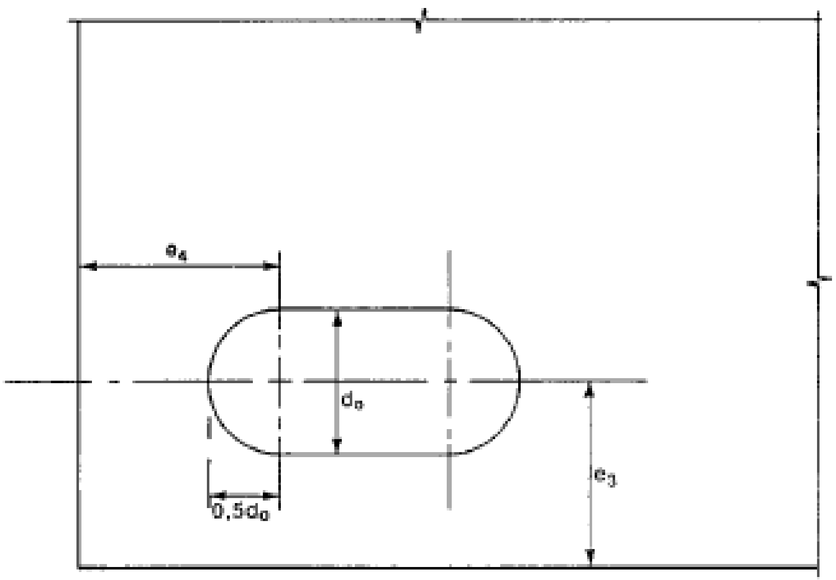


Figure 3.1: Symbols for end and edge distances and spacing of fasteners



e) End and edge distances for slotted holes

Figure 3.1: Symbols for end and edge distances and spacing of fasteners

Design Resistance of Individual Fasteners

(EC3-1-8, Cl 3.6)

1. The design resistance for an individual fastener subjected to shear, tension or bearing forces, or any combination of these forces, is given in Table 3.4.

2. The design resistances for tension and for shear through the threaded portion of a bolt given in Table 3.4 should only be used for bolts manufactured in conformity with the standards listed in Cl 1.2.4.

Table 3.4: Design resistance for individual fasteners subjected to shear and/or tension

Failure mode	Bolts	Rivets
Shear resistance per shear plane	$F_{v,Rd} = \dfrac{\alpha_v \; f_{ub} \; A}{\gamma_{M2}}$ - where the shear plane passes through the threaded portion of the bolt (A is the tensile stress area of the bolt A_s): - for classes 4.6, 5.6 and 8.8: $\alpha_v = 0{,}6$ - for classes 4.8, 5.8, 6.8 and 10.9: $\alpha_v = 0{,}5$ - where the shear plane passes through the unthreaded portion of the bolt (A is the gross cross section of the bolt): $\alpha_v = 0{,}6$	$F_{v,Rd} = \dfrac{0{,}6 \; f_{ur} \; A_0}{\gamma_{M2}}$
Bearing resistance [1), 2), 3)]	AC2 $F_{b,Rd} = \dfrac{k_1 \alpha_b \, f_u \, d \, t}{\gamma_{M2}}$ AC2 where α_b is the smallest of α_d ; $\dfrac{f_{ub}}{f_u}$ or 1,0; in the direction of load transfer: - for end bolts: $\alpha_d = \dfrac{e_1}{3d_0}$; for inner bolts: $\alpha_d = \dfrac{p_1}{3d_0} - \dfrac{1}{4}$ perpendicular to the direction of load transfer: AC2 - for edge bolts: k_1 is the smallest of $2{,}8\dfrac{e_2}{d_0} - 1{,}7 , 1{,}4\dfrac{p_2}{d_0} - 1{,}7$ and 2,5 AC2 - for inner bolts: k_1 is the smallest of $1{,}4\dfrac{p_2}{d_0} - 1{,}7$ or 2,5	

Tension resistance [2]	$F_{t,Rd} = \dfrac{k_2\ f_{ub}\ A_s}{\gamma_{M2}}$ where $k_2 = 0{,}63$ for countersunk bolt, otherwise $k_2 = 0{,}9$.	$F_{t,Rd} = \dfrac{0{,}6\ f_{ur}\ A_0}{\gamma_{M2}}$
Punching shear resistance	$B_{p,Rd} = 0{,}6\ \pi\ d_m\ t_p\ f_u\ /\ \gamma_{M2}$	No check needed
Combined shear and tension	$\dfrac{F_{v,Ed}}{F_{v,Rd}} + \dfrac{F_{t,Ed}}{1{,}4\,F_{t,Rd}} \le 1{,}0$	

[1] The bearing resistance $F_{b,Rd}$ for bolts

- in oversized holes is 0,8 times the bearing resistance for bolts in normal holes.
- in slotted holes, where the longitudinal axis of the slotted hole is perpendicular to the direction of the force transfer, is 0,6 times the bearing resistance for bolts in round, normal holes.

[2] For countersunk bolt:

- the bearing resistance $F_{b,Rd}$ should be based on a plate thickness t equal to the thickness of the connected plate minus half the depth of the countersinking.
- for the determination of the tension resistance $F_{t,Rd}$ the angle and depth of countersinking should conform with 1.2.4 Reference Standards: Group 4, otherwise the tension resistance $F_{t,Rd}$ should be adjusted accordingly.

[3] When the load on a bolt is not parallel to the edge, the bearing resistance may be verified separately for the bolt load components parallel and normal to the end.

3. The design shear resistance $F_{v,Rd}$ given in Table 3.4 should only be used where the bolts are used in holes with nominal clearances not exceeding those for normal holes as specified in Cl 1.2.7. The nominal clearance in standard holes for **bolted connections:**

1 mm for M12 and M14 bolts.
2 mm for M16 to M24 bolts.
3 mm for M27 and larger bolts.

The nominal clearance in oversize holes for **slip-resistant connections:**

3 mm for M12 bolts.
4 mm for M14 to M22 bolts.
6 mm for M24 bolts.
8 mm for M27 and larger bolts.

4. The partial safety factors, γ_M for joints are given in Table 2.1, Cl 2.2(2).

Table 2.1: Partial safety factors for joints

Resistance of members and cross-sections	γ_{M0} , γ_{M1} and γ_{M2} see EN 1993-1-1
Resistance of bolts	γ_{M2}
Resistance of rivets	
Resistance of pins	
Resistance of welds	
Resistance of plates in bearing	
Slip resistance - at ultimate limit state (Category C) - at serviceability limit state (Category B)	 γ_{M3} $\gamma_{M3,ser}$
Bearing resistance of an injection bolt	γ_{M4}
Resistance of joints in hollow section lattice girder	γ_{M5}
Resistance of pins at serviceability limit state	$\gamma_{M6,ser}$
Preload of high strength bolts	γ_{M7}
Resistance of concrete	γ_c see EN 1992

NOTE: Numerical values for γ_M may be defined in the National Annex. Recommended values are as follows: $\gamma_{M2} = 1,25$; $\gamma_{M3} = 1,25$ and $\gamma_{M3,ser} = 1,1$; $\gamma_{M4} = 1,0$; $\gamma_{M5} = 1,0$; $\gamma_{M6,ser} = 1,0$; $\gamma_{M7} = 1,1$.

5. The following tables summarized the capacities of class 8.8 ordinary bolts of steel design grades S275 and S355.

Capacities of class 8.8 ordinary bolts: design grade S275 material

Bolt size	Tensile stress area	Tensile capacity at 533N/mm^2	Shear capacity at 355 N/mm^2 threads in shear plane		Bearing capacity at 430N/mm^2 foredge distance e = 2.0 and spacing p = 3.5d							
					Thickness – mm – plate passed through							
	mm^2	kN	Single	Double	6	8	10	12	15	20	25	30
M16	157	83.7	55.7	111	41.3	55.1	68.8	82.6	103	138		
M20	245	131	87.0	174	51.6	68.8	86.0	103	129	172		
M24	353	188	125	251	61.9	82.5	103	124	155	206	258	
M30	561	299	199	398	77.4	103	129	155	194	258	323	387

Capacities of class 8.8 ordinary bolts: design grade S355 material

Bolt size	Tensile stress area	Tensile capacity at $533 N/mm^2$	Shear capacity at $355 N/mm^2$ threads in shear plane		Bearing capacity at $510 N/mm^2$ for edge distance e = 2.0 and spacing p = 3.5d Thickness – mm – plate passed through						
	mm^2	kN	Single	Double	6	8	10	12	15	20	25
M16	157	83.7	55.7	111	49.0	65.3	61.6	97.9	122		
M20	245	131	87.0	174	61.2	81.6	102	122	153	204	
M24	353	188	125	251	73.4	97.9	122	147	184	245	
M30	561	299	199	398	91.8	122	153	184	230	306	383

6. In single lap joints with only one bolt row (see Figure 3.3), the bolts should be provided with washers under both the head and the nut. The design bearing resistance $F_{b,Rd}$ for each bolt should be limited to [eqn. (3.2)]:

$$F_{b,Rd} \leq 1.5\, f_u\, d\, t / \gamma_{M2}$$

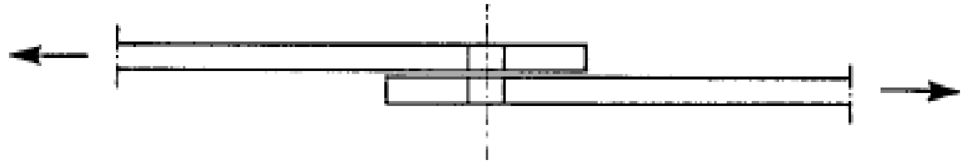


Figure 3.3: Single lap joint with one row of bolts

7. In the case of class 8.8 or 10.9 bolts, hardened washers should be used for single lap joints with only one bolt or one row of bolts.

8. Where bolts transmitting load in shear and bearing pass through packing of total thickness t_p greater than one-third of the nominal diameter d (see Figure 3.4), the design shear resistance $F_{v,Rd}$ calculated as specified in Table 3.4 should be multiplied by a reduction factor β_p given by [eqn. (3.3)]:

$$\beta_p = \frac{9d}{8d + 3t_p} \qquad \text{but} \qquad \beta_p \leq 1$$

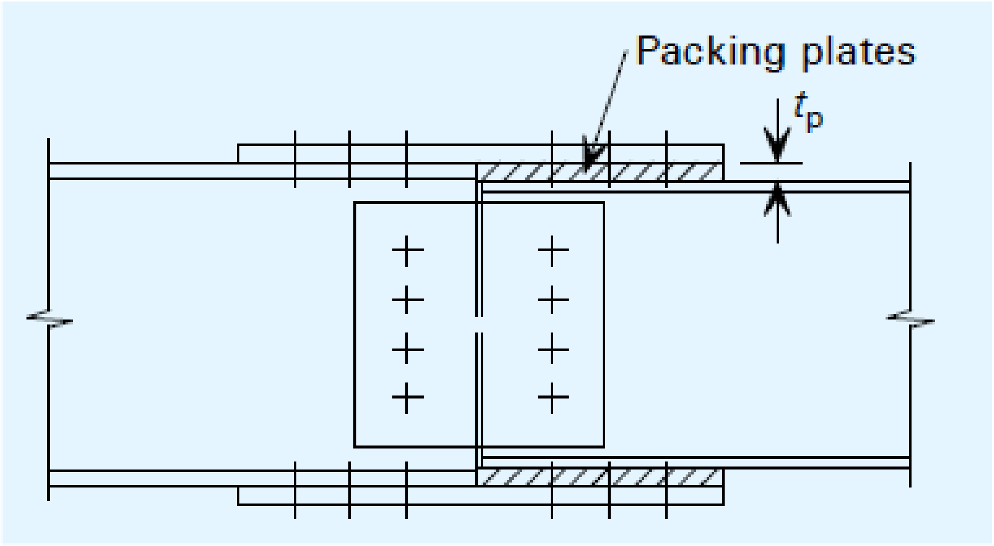


Figure 3.4 Fasteners through packings

Although no upper limit to packing thickness is given in EC3-1-8, it is recommended that the thickness should not exceed 4d/3, as stated in BS 5950-1.

9. For double shear connections with packing on both sides of the splice t_p should be taken as the thickness of the thicker packing.

Group of Fasteners

(EC3-1-8, Cl 3.7)

1. The design resistance of a group of fasteners may be taken as the sum of the design bearing resistances $F_{b,Rd}$ of the individual fasteners provided that the design shear resistance $F_{v,Rd}$ of each individual fastener is greater than or equal to the design bearing resistance $F_{b,Rd}$.

2. Otherwise, the design resistance of a group of fasteners should be taken as the number of fasteners multiplied by the smallest design resistance of any of the individual fasteners.

3. In any group of fasteners, the bearing resistances may vary (due to the end distance affecting the resistance of the end bolts). The shear resistance will be the same for each bolt. Consider three cases:

- Case (i) The shear bearing resistances are less than the shear resistance – the resistance of the group is the sum of the individual bearing resistances.
- Case (ii) The shear resistance is greater than some (but not all) bearing resistances in the group – the resistance of the connection is the minimum bearing resistance in the group, multiplied by the number of bolts.
- Case (iii) The shear resistance is less than all the bearing resistances in the group – the resistance of the connection is the shear resistance of a bolt, multiplied by the number of bolts.

High Strength Bolts in Slip-Resistant Connections

(EC3-1-8, Cl 3.9)

1. High strength bolts a.k.a high strength friction grip (HSFG) bolts can be preloaded when installed and design to be slip-resistant at working load or ultimate load.

2. The design slip resistance of a preloaded bolt of class 8.8 or 10.9 should be taken as [eqn. (3.6)]:

$$F_{s,Rd} = \frac{k_s \; n \; \mu}{\gamma_{M3}} F_{p,C}$$

or

$$F_{s,Rd,ser} = \frac{k_s n \mu}{\gamma_{M3,ser}} F_{p,C}$$

where,

k_s is given in Table 3.6.
n is the number of the friction planes.
μ is the slip factor obtained either by specific tests for the friction surface in accordance with Cl 1.2.7 Reference Standards: Group 7 or when relevant as given in Table 3.7.

Table 3.6: Values of k_s

Description	k_s
Bolts in normal holes.	1,0
Bolts in either oversized holes or short slotted holes with the axis of the slot perpendicular to the direction of load transfer.	0,85
Bolts in long slotted holes with the axis of the slot perpendicular to the direction of load transfer.	0,7
Bolts in short slotted holes with the axis of the slot parallel to the direction of load transfer.	0,76
Bolts in long slotted holes with the axis of the slot parallel to the direction of load transfer.	0,63

Table 3.7: Slip factor, μ, for pre-loaded bolts

Class of friction surfaces (see 1.2.7 Reference Standard: Group 7)	Slip factor μ
A	0,5
B	0,4
C	0,3
D	0,2

NOTE 1: The requirements for testing and inspection are given in 1.2.7 Reference Standards: Group 7.

NOTE 2: The classification of any other surface treatment should be based on test specimens representative of the surfaces used in the structure using the procedure set out in 1.2.7 Reference Standards: Group 7.

NOTE 3: The definitions of the class of friction surface are given in 1.2.7 Reference Standards: Group 7.

NOTE 4: With painted surface treatments a loss of pre-load may occur over time.

2. For class 8.8 and 10.9 bolts conforming with Cl 1.2.4 Reference Standards: Group 4, with controlled tightening in conformity with Cl 1.2.7 Reference Standards: Group 7, the preloading force $F_{p,C}$ to be used in eqn. (3.6) should be taken as [eqn. (3.7)]:

$$\mathbf{F_{p,C} = 0.7\, f_{ub}\, A_s}$$

3. The capacities of class 10.9 ordinary bolts of design steel grade S355 are summarized in the table below.

Bolt size	Tensile stress area	Tensile capacity at 667N/mm^2	Shear capacity at at 370N/mm^2 threads in shear plane		Bearing capacity at 1020N/mm^2 for edge distance e = 2.0 and spacing p = 3.5d				
					Thickness – mm – plate passed through				
	mm^2	kN	Single	Double	6	8	10	12	15
M16	157	105	58.1	1161	98.0	131			
M20	245	163	90.7	181	122	163	204		
M24	353	235	131	161	147	196	245	294	
M30	561	374	208	416	184	245	306	367	459

Block Tearing

(EC3-1-8, Cl 3.10.2)

1. Block tearing consists of failure in shear at the row of bolts along the shear face of the hole group accompanied by tensile rupture along the line of bolt holes on the tension face of the bolt group. Figure 3.8 shows block tearing.

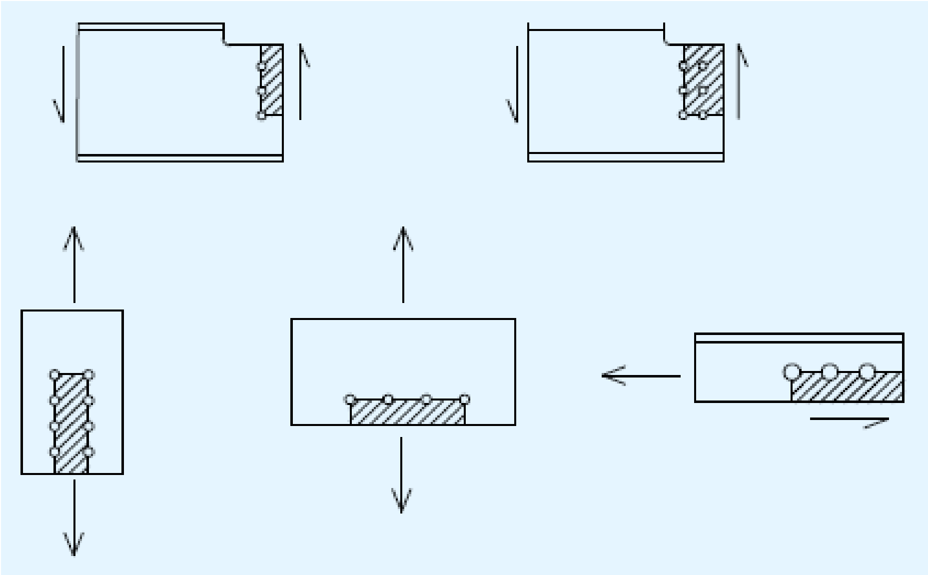


Figure 3.8 Examples of block tearing

2. Note that in Figure 3.8, two examples show bolt groups subject to concentric loading and the remainder show bolt groups subject to eccentric loading.

3. For a symmetric bolt group subject to concentric loading, the design block tearing resistance is given by [eqn. (3.9)]:

$$V_{\mathrm{eff,1,Rd}} = \frac{f_u A_{nt}}{\gamma_{M2}} + \frac{(1/\sqrt{3}) f_y A_{nv}}{\gamma_{M0}}$$

where A_{nt} is net area subject to tension.
A_{nv} is net area subject to shear.

4. For a bolt group subject to eccentric loading, the design block shear tearing resistance is given by [eqn. (3.10)]:

$$V_{\mathrm{eff,2,Rd}} = \frac{0.5 f_{\mathrm{u}} A_{\mathrm{nt}}}{\gamma_{\mathrm{M2}}} + \frac{(1/\sqrt{3}) f_{\mathrm{y}} A_{\mathrm{nv}}}{\gamma_{\mathrm{M0}}}$$

WELDED CONNECTIONS

3. Generally in connections, plates intersect at right angles but intersection angles of between 60° and 120° can be used provided that the correct throat size is used in design calculations.

4. In order to accommodate lack of fit, the minimum leg length *S* of fillet weld is 5 mm although 6 mm is often preferred to guarantee quality. The maximum size of fillet weld from a single run metal arc process is 8 mm. When larger fillet welds are required they are formed from multiple runs.

5. The use of intermittent fillet weld is permitted (Cl 4.3.2.2) but not favoured because they introduce stress discontinuities, act as stress raisers, may introduce fatigue cracks, may act as corrosion pockets and are difficult to produce with an automatic welding machine.

6. The size of a weld is often described by the leg length but the strength is calculated using the effective throat thickness *a* (Cl 4.5.2), which should not be less than 3 mm.

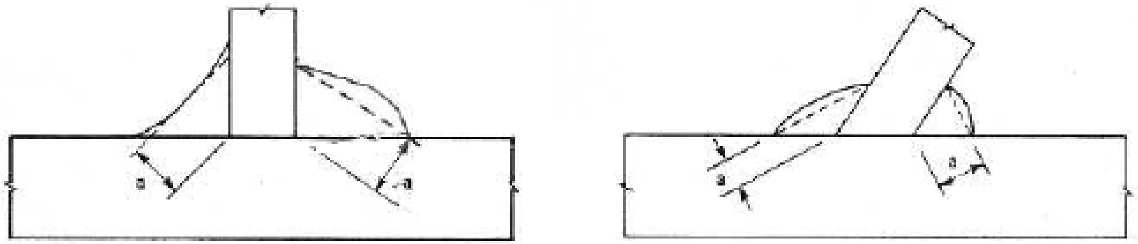


Figure 4.3: Throat thickness of a fillet weld

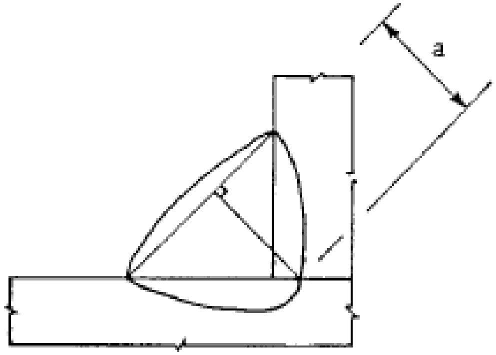
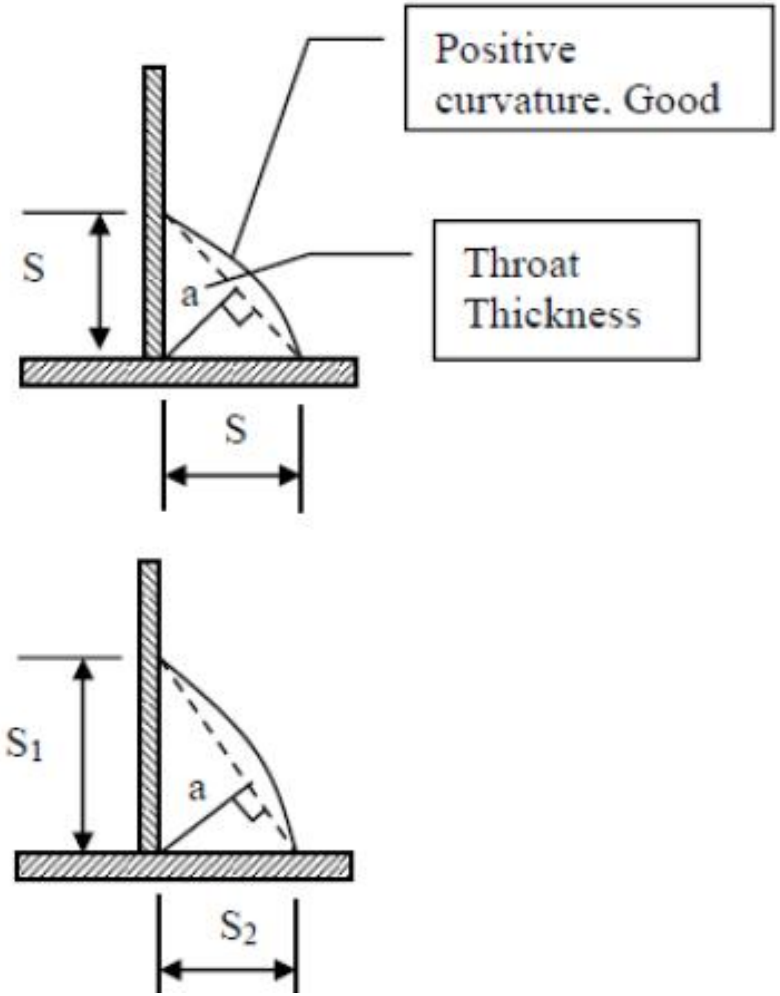


Figure 4.4: Throat thickness of a deep penetration fillet weld

Figure 4.4: Throat thickness of a deep penetration fillet weld



Case (a) Equal Leg Fillet

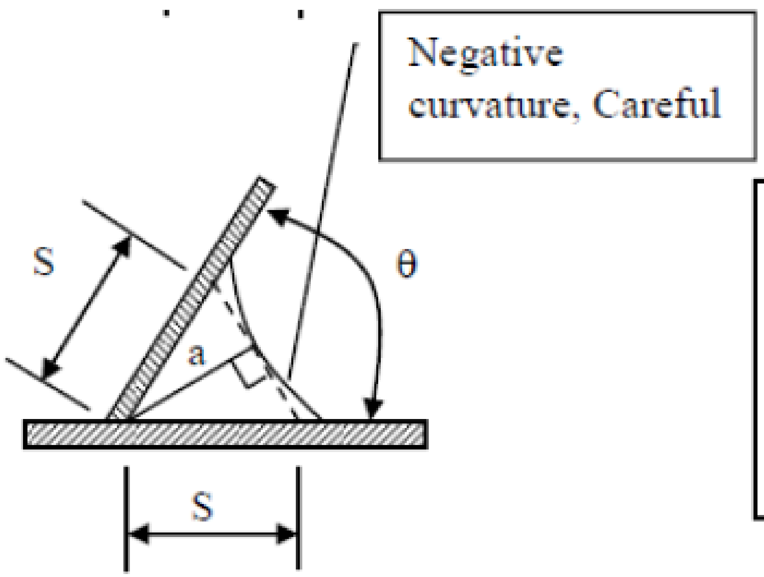
$a = s\cos(45) < 0.7s$

Case (b) Unequal Leg Fillet

$a = s_2\cos(\theta) < 0.7s$

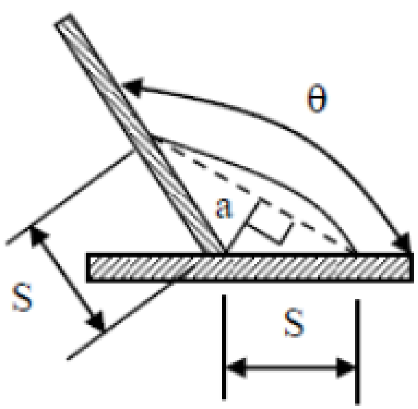
where $\tan(\theta) = S_2/S_1$.

Figure 4.4: Throat thickness of a deep penetration fillet weld



Case (c) Acute Angled Fillet

$$a = s\cos\left(\frac{\theta}{2}\right) < 0.7s$$



Case (d) Obtuse Angled Fillet

$$a = s\cos\left(\frac{\theta}{2}\right) < 0.7s$$

7. The effective length of a fillet weld l_{eff} (Cl 4.5.1) should be taken as the length over which the fillet is full size. This may be taken as the overall length of the weld reduced by twice the effective throat thickness i.e.

$$l_{eff} = \mathbf{L - 2}\boldsymbol{a}$$

8. The minimum length of the weld allowed to transmit loading is **6a or 30 mm** whichever is the larger.

9. There are two methods recommended in EC3-1-8 for the design of fillet welds namely Directional Method (Cl 4.5.3.2) and Simplified Method (Cl 4.5.3.3). However, only the latter will be discussed here.

The Simplified Method

(EC3-1-8, Cl 4.5.3.3)

1. The design resistance of the fillet weld will be adequate if at all points the design value of the weld force per unit length $F_{w,Ed}$ is less than or equal to the design weld resistance per unit length $F_{w,Rd}$ i.e.

$$\mathbf{F_{w,Ed} \leq F_{w,Rd}} \qquad \text{(eqn. 4.2)}$$

2. The design weld resistance per unit length of a fillet weld is given by:

$$\mathbf{F_{w,Rd} = f_{vw,Rd}\, \boldsymbol{a}} \qquad \text{(eqn. 4.3)}$$

where a = throat thickness

$f_{vw,Rd}$ = the design shear strength of the weld

3. The design shear strength of the weld should be determined from:

$$f_{vw,Rd} = \frac{f_u / \sqrt{3}}{\beta_w \gamma_{M2}} \qquad \text{(eqn. 4.4)}$$

where f_u = nominal ultimate tensile strength of the weaker part joined (EC3-1-1, Table 3.1)

β_w = the appropriate correlation factor taken from Table 4.1.

Table 4.1: Correlation factor β_w for fillet welds

Standard and steel grade			Correlation factor β_w
EN 10025	**EN 10210**	**EN 10219**	
S 235 S 235 W	S 235 H	S 235 H	0,8
S 275 S 275 N/NL S 275 M/ML	S 275 H S 275 NH/NLH	S 275 H S 275 NH/NLH S 275 MH/MLH	0,85
S 355 S 355 N/NL S 355 M/ML S 355 W	S 355 H S 355 NH/NLH	S 355 H S 355 NH/NLH S 355 MH/MLH	0,9
S 420 N/NL S 420 M/ML		S 420 MH/MLH	1,0
S 460 N/NL S 460 M/ML S 460 Q/QL/QL1	S 460 NH/NLH	S 460 NH/NLH S 460 MH/MLH	1,0

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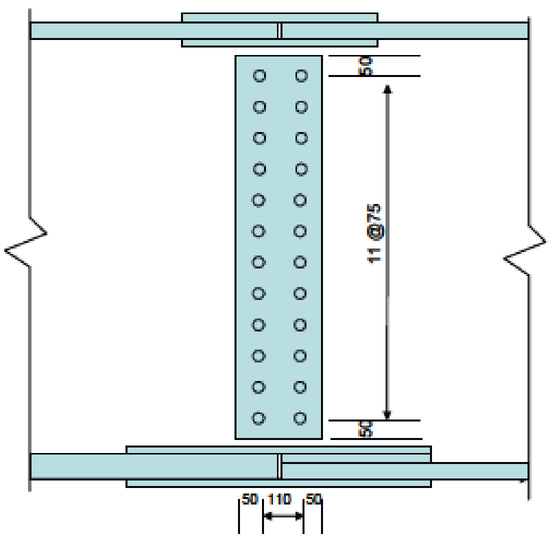
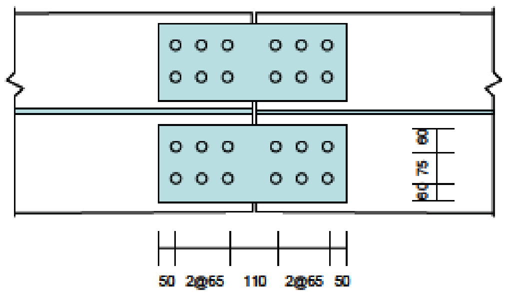
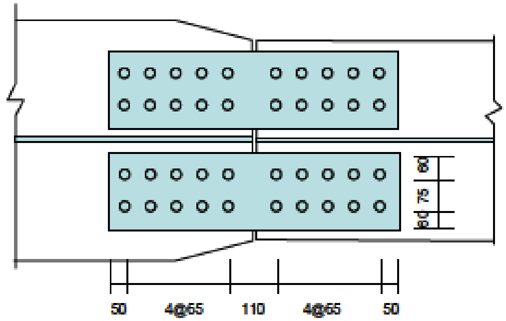
Verify the adequacy of the cross-section resistance given in Figure below



Elevation and web splice



Top flange (lower cover plates)



Bottom flange (upper cover plates)

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12 Main girder splices

12.1 Forces and moments at splice position

Design effects to be considered:

Worst hogging moment at splice, at ULS and SLS

Worst shear at splice, at ULS and SLS

(The worst sagging moment is much less than maximum hogging moment, as noted before) Sheet 25

Consider the stresses in the pier girder side of the splice. The stress distribution will be different on the span girder side of the splice but the total moments and forces at the splice position must be the same. Because the bottom flange is smaller, more force will be carried in the web on the span side but since the moment on the bolt group on the pier side is increased by its eccentricity from the centreline of the splice and the moment on the group on the span side is decreased, it can be shown that the total effects on the bolt group are less on the span side. A symmetric arrangement of bolts, designed for the pier side, will thus be satisfactory.

The in-service design combinations of actions considered are:

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The in-service design combinations of actions considered are:

(1) Construction load + traffic load for worst hogging + force due to temperature expansion

(2) Construction load + traffic load for worst shear + force due to temperature expansion.

	ULS hog	SLS hog	ULS shear	SLS shear
Top flange stress	23	21	−38	−23 N/mm²
Bottom flange stress	−110	−88	−10	−14 N/mm²
Shear force	834	650	1320	1011 N/mm²

From the above stresses, the forces in each flange, the axial force and moment in the web are as follows:

	ULS hog	SLS hog	ULS shear	SLS shear
Top flange force	460	420	−760	−460 kN
Bottom flange force	−3960	−3168	−360	−504 kN
Web force	−644	−497	−336	−259 kN
Web moment	161	132	−33	−11 kNm

Actions at the construction stage

It is noted that the compressive stress in the top flange is higher at construction stage 1, under wet concrete load in span 1. At that stage, the splice must provide continuity of stiffness, without slipping, and because the beams are slender at that stage it is appropriate to amplify the design force to ensure adequate continuity of resistance.

The maximum stress in the top flange during construction is 42 N/mm² — Sheet 23

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Sheet 30

The slenderness of the beam at that stage is $\bar{\lambda}_{LT} = 0.89$ which means that

$M_{c,Rk}/M_{cr} = 0.89^2 = 0.79$

The midspan bending moment at the bare steel stage is $M_{Ed} = 3132$ kNm (Sheet 23) and the resistance of the cross section $M_{c,Rk} = 2.287 \times 10^7 \text{ mm}^3 \times 345 = 7890$ kNm.

Hence $\dfrac{M_{Ed}}{M_{cr}} = \dfrac{M_{c,Rk}}{M_{cr}} \times \dfrac{M_{Ed}}{M_{c,Rk}} = 0.79 \times \dfrac{3132}{7890} = 0.316$

Second order effects can thus be determined by multiplying by $\dfrac{1}{(1-0.316)} = 1.46$

Thus the design force for the top flange is $840 \times 1.46 = 1226$ kN

(Clearly this is more onerous than in the final situation, where the stresses are of similar magnitude but the flange is restrained against buckling and no amplification is needed.)

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12.2 Slip resistance of bolts	
Use M24 grade 8.8 preloaded bolts in double shear in normal clearance holes with class A friction surface:	3-1-8/3.9.1
$d = 24$ mm $d_0 = 26$ mm $f_{ub} = 800$ N/mm^2 $A_s = 353$ mm^2 $\mu = 0.5$ $k_s = 1.0$	
Preload force $F_{p,C} = 0.7 f_{ub} A_s = 0.7 \times 800 \times 353 \times 10^{-3} = 198$ kN	
ULS Slip resistance of bolts (double shear)	
$F_{s,Rd} = \dfrac{k_s n \mu}{\gamma_{M3}} F_{p,C} = \dfrac{1.0 \times 2 \times 0.5}{1.25} \times 198 = 158\,\text{kN}$	
For SLS slip resistance, use the same equation but divide by $\gamma_{M3,ser}$ (= 1.1)	
Slip resistance in double shear = 198/1.1 = 180 kN	
12.3 Shear resistance of bolts	
ULS shear resistance of bolt (assuming shear through threads):	3-1-8/ Table 3.4
$F_{v,RD} = \dfrac{\alpha_v f_{ub} A_s}{\gamma_{M2}} = \dfrac{0.6 \times 800 \times 353}{1.25} = 136\,\text{kN}$	

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12.4 Bolt spacing and edge distances

Limiting spacings for M24 bolts, for strength: — 3-1-8/ Table 3.3

End and edge distances: $1.2d_0 = 1.2 \times 26 = 31.2$ mm
Spacing in direction of force: $2.2d_0 = 2.2 \times 26 = 57.2$ mm
Spacing perpendicular to force $2.4d_0 = 2.4 \times 26 = 62.4$ mm

Limiting spacings for M24 bolts, for fatigue classification: — 3-1-9/ Table 8.1

End and edge distances: $1.5d = 1.5 \times 26 = 39$ mm
Spacing: $2.5d = 2.5 \times 24 = 60$ mm

(The parameter d is not specified in Table 8.1 but GN 5.08 (P185[6]), suggests use of hole diameter for edge distances and bolt diameter for spacings.)

For detailing purposes, use minima of 40 mm, 65 mm and 70 mm respectively

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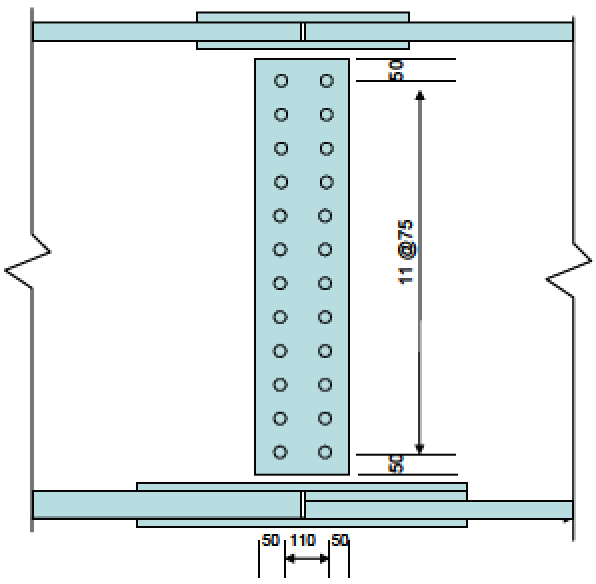
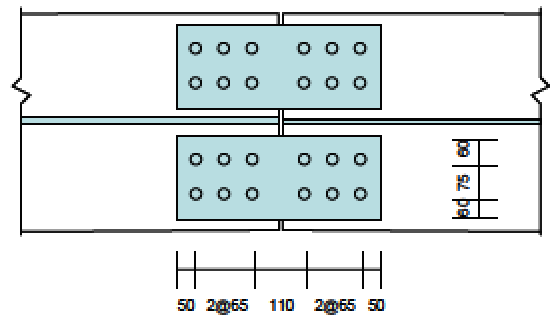
12.5 Splice configuration

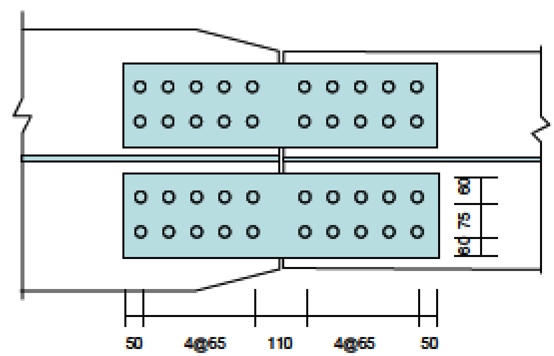
Consider the following splice configuration:



Elevation and web splice



Top flange (lower cover plates)



Bottom flange (upper cover plates)

TUTORIAL 2 (refer to hand out)

Top flange splice

(Dimensions for lower covers)

Bolt spacing:
In line of force: $e_1 = 50$ mm, $p_1 = 65$ mm
Perpendicular to force: $e_2 = 60$ mm, $p_2 = 75$ mm
Overall dimension 470 × 195 mm
Thickness 10 mm

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The length of the cover is sufficiently short that stud shear connectors do not need to be welded to the upper cover (the maximum permitted longitudinal spacing is 800 mm).	4-2/6.6.5.5

Bottom flange splice

(Dimensions for upper covers)

Bolt spacing:

In line of force: $e_1 = 50$ mm, $p_1 = 65$ mm

Perpendicular to force: $e_2 = 60$ mm, $p_2 = 75$ mm

Overall dimension 210 × 195 mm.

Thickness 20 mm

Web splice

Bolt spacing:

In line of force: $e_1 = 50$ mm (only a single column, so no p_1 value)

Perpendicular to force: $e_2 = 50$ mm, $p_2 = 75$ mm

Overall dimension 730 × 925 mm.

Thickness 10 mm

The web depth on the support side of the splice is 1000 mm and if the web splice is positioned symmetrically within this depth, the centreline of the lowest bolt will be 87.5 mm above the flange and 67.5 mm above the cover plate. This is adequate for the tightening of the bolt (see GN 2.06, P185[6]).

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12.6 Verification of connection resistances

Top flange splice

There are 3 rows of bolts, with 4 bolts per row across the flange.

A category C connection is required (the design situation is for resistance against buckling of the beam during construction).

Slip resistance at ULS $= 12 \times 158 = 1896$ kN > 1226 kN adequate

Bottom flange splice

There are 5 rows of bolts, with 4 bolts per row across the flange.

A category B connection is required (the design situation is for resistance against compression in the flange in service).

Resistance at ULS

ULS slip resistance $= 20 \times 158 = 3160$ kN < 3960 kN so the splice will slip into bearing at ULS

ULS shear resistance of bolt group $= 20 \times 272 = 5440$ kN - adequate

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TUTORIAL 2 (refer to hand out)

ULS bearing resistance per bolt is given by:

$$F_{b,Rd} = \frac{k_1 \alpha_b f_u d t}{\gamma_{M2}} =$$

Bolt spacings, for determination of factors k_1 and α_b

In line of force: $e_1 = 50$ mm, $p_1 = 65$ mm

Perpendicular to force: $e_2 = 60$ mm, $p_2 = 75$ mm

Since $f_{ub} > f_u$, $\alpha_b = \alpha_d$ (but ≤ 1)

For end bolts: $\alpha_d = e_1/3d_0 = 50/(3 \times 26) = 0.64$

For inner bolts: $\alpha_d = p_1/d_0 - ¼ = 65/(3 \times 26) - 0.25 = 0.58$

For edge bolts k_1 is the smaller of $2.8e_2/d_0 - 1.7$ and 2.5

$k_1 = \min(2.8 \times 60/26 - 1.7; 2.5) = 2.5$

In the upper cover plates there is no 'inner' line of bolts (in the direction of force) and for the flange and lower cover, the mean value of p_2 that would apply is sufficient to ensure that $k_1 = 2.5$

The value of f_u is given by the product standard for S355 plates as 470 kN/mm^2 — EN 10025-2

Conservatively, using $\alpha_b = 0.58$ the resistance of the bolt in 20 mm covers is:

$$F_{b,Rd} = \frac{2.50 \times 0.58 \times 470 \times 24 \times 20}{1.25} = 262 \text{ kN}$$

Bearing resistance of group, with double covers $= 20 \times 2 \times 262 = 10480$ kN

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The ULS bearing resistance is adequate and the connection resistance is determined by the shear resistance of the bolts. Note that, on the span side, 20 mm packing is used. This would reduce the bearing/shear resistance on the upper shear plane by about 15% (see 3–1–8/3.6.1(12)) but the resistance would still be adequate.

Resistance at SLS

SLS slip resistance of group $= 20 \times 180 = 3600$ kN > 3312 kN satisfactory

Web splice

The splice has a single column of 12 bolts at 75 mm spacing

For this group the 'modulus' for the outer bolts $= \sum r_i^2 / r_{max}$

where r_i is the distance of each bolt from the centre of the group and r_{max} is the distance of the furthest bolt.

Here, the modulus $= 1950$ mm

The extra moment due to the shear $=$ shear force $\times$ eccentricity of group from the centreline of the splice

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Hence the force on the outer bolts at ULS and SLS are

	ULS hog	SLS hog	ULS shear	SLS shear	
Shear V	834	650	1320	1011 kN	
Longitudinal force F_L	609	469	–336	–259 kN	
Moment	155	127	–33	–11 kNm	
Moment due to e = 55 mm	46	36	73	56 kNm	
Total Moment M	201	163	40	45 kNm	
Force per bolt due to M	103	84	21	23 kN	(= M/1950)
Force per bolt due to F_L	51	39	–28	22 kN	(= F_L/12)
Total horizontal force	154	123	49	45 kN	
Vertical force due to V	70	54	110	84 kN	(= V/12)
Resultant force	169	134	120	95 kN	(Vector sum)

Bearing resistance for web bolts

Note: The directions of the resultant forces are not parallel to an edge. Table 3.4 suggests that in such cases the parallel and normal components could be verified separately but no interaction relationship is suggested. Here the direction of the resultant force being not normal to the long edge of the cover plate edge is considered not to have an adverse effect on the factors, since the edge distance is less than the end distance. The factors for resistance in a horizontal direction are therefore used.

For end bolts (there is only a single row, transverse to the force):

$\alpha_d = e_1/3d_0 = 50\ /(3 \times 26) = 0.64$

For edge bolts $k_1 = \min(2.8 \times 50/26 - 1.7;\ 2.5) = 2.5$

For inner bolts $k_1 = \min(1.4 \times 75/26 - 1.7;\ 2.5) = 2.34$

With two 10 mm covers, the bearing stress on the 14 mm web is higher (and is higher again on the 10 mm web, although the values for the design forces on that side of the splice are lower and are not shown here)

TUTORIAL 2 (refer to hand out)

$$F_{b,Rd} = \frac{2.50 \times 64 \times 470 \times 24 \times 14}{1.25} = 202 \text{ kN (for end bolts, 192 kN for inner bolts)}$$

The bearing resistance is less than the resistance of the bolts in double shear (272 kN), so bearing resistance governs.

The maximum resultant force at ULS (164 kN) exceeds the slip resistance (158 kN) but is less than the resistance in bearing and shear (202 kN) therefore the bolt group is satisfactory. The maximum force at SLS is 134 kN and the resistance is 180 kN so there is no slip at SLS. The forces on the inner bolts are less and are satisfactory by inspection.

12.7 Forces in cover plates

The cover plates are verified as members in tension or compression, in accordance with EN 1993-1-1.

TUTORIAL 2 (refer to hand out)

Top flange	
The covers are in tension. Assume half of the load is carried in the lower cover plates. The force per cover plate thus = 1302/4 = 326 kN	
Area of gross cross section = 195 × 10 = 1950 mm²	
Area of net section = 1950 − 2 × 26 × 10 = 1430 mm²	
This is a Category C slip resistant connection, therefore the design tension resistance is given by:	
$N_{net,Rd} = \dfrac{A_{net} f_y}{\gamma_{M0}} = \dfrac{1430 \times 355}{1.0} \times 10^{-3} = 508 \text{ kN}$ Satisfactory	3-1-1/6.2.3
The maximum spacing of bolts is 110 mm and the limiting spacing is given by Table 3.3 as the smaller of 14t (= 140 mm) and 200 mm. Since p_1/t = 65/20 = 3.25, which is less than 9ε (=7.2) buckling does not need to be checked. The spacing is satisfactory.	3-1-1/ Table 3.3
Bottom flange	
The covers are in compression. Assume half of the load is carried in the upper cover plates. The force per cover plate thus = 3960/4 = 990 kN	
Fastener holes do not need to be deducted (unless oversize holes are allowed), therefore A = 1950 × 20 = 3900 mm²	
$N_{pl,Rd} = \dfrac{A f_y}{\gamma_{M0}} = \dfrac{3900 \times 345}{1.0} \times 10^{-3} = 1346 \text{ kN}$ Satisfactory	
The maximum spacing of bolts is 110 mm and the limiting spacing is given by Table 3.3 as the smaller of 14t (= 280 mm) and 200 mm. The spacing is satisfactory.	3-1-1/ Table 3.3

TUTORIAL 2 (refer to hand out)

Web Consider the stresses in the cover plate on a line through the vertical row of bolts. The moment on each cover plate = 201/2 = 101 kN The axial force = 609/2 = 305 kN The shear force = 834/2 = 417 kN The stress at the bottom of the cover plate is thus: $(101 \times 10^6)/(10 \times 925^2/6) + (305\times 10^3)/(925 \times 10)$ $= 71 + 33 = 104$ N/mm^2	
The value of $p_1/t = 115/10 = 11$, which is greater than 9ε $(=7.2)$ so buckling must be checked. Using a buckling length of $0.6p_1 = 66$ mm and $i = 10/\sqrt{12} = 2.89$ mm, the slenderness is: $\lambda = \dfrac{L_{cr}}{i\lambda_1} = \dfrac{66}{2.98 \times 76.5} = 0.30$	3-1-8/ Table 3.3 3-1-1/6.3.1.3

TUTORIAL 2 (refer to hand out)

SCI Silwood Park, Ascot, Berks SL5 7QN Telephone: (01344) 636525 Fax: (01344) 636570 CALCULATION SHEET	Job No. BCR113		Sheet 57 of 64		Rev A
	Job Title Composite highway bridges: Worked examples				
	Subject Example 1: Multi-girder two-span bridge Section 12: Main girder splices				
	Client SCI	Made by DCI		Date July 2009	
		Checked by JMS		Date Sep 2009	

From buckling curve a, $\chi = 0.98$, so the limiting stress $= 0.98 \times 355/1.1 = 316$ N/mm^2, which is satisfactory. The spacing also complies with the limit of $14t$ ($= 140$ mm).

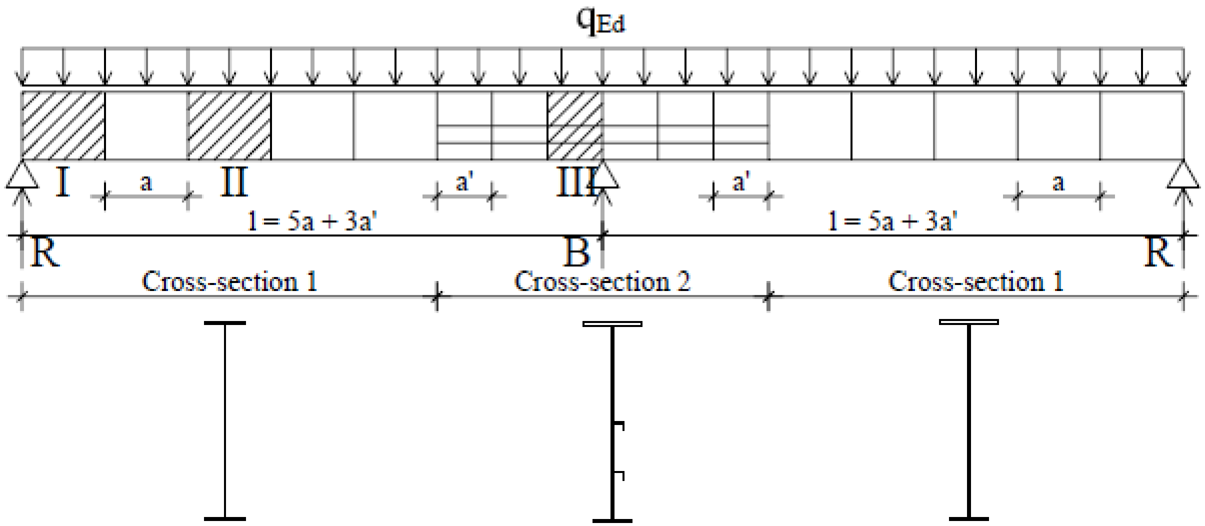
The shear stress is:

$417 \times 10^3 / (10 \times 925) = 45$ N/mm^2

This is satisfactory and is low enough that the resistance to direct stress does not need to be reduced.

SELF ASSESSMENT

- Verify overall adequacy of the plate girder below



SELF ASSESSMENT

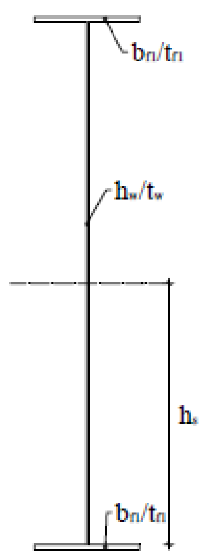
$S\ 235 \Rightarrow f_y = 235\ \text{MPa}$

$$\varepsilon = \sqrt{\frac{235}{f_y\left[N/mm^2\right]}} = \sqrt{\frac{235}{235}} = 1$$

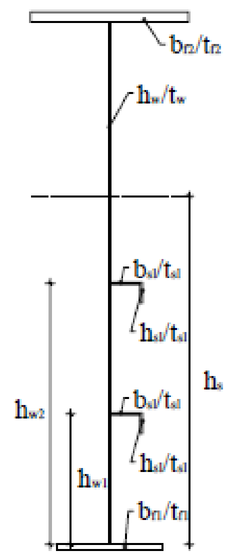
$\gamma_{M0} = 1{,}0$

$\gamma_{M1} = 1{,}0$

Cross-section 1



Cross-section 2



$a\ \ = 3\ \text{m}$

$a'\ = 2\ \text{m}$

$l\ \ = 21\ \text{m}$

$q_{Ed} = 85\ \text{kN/m}$

$b_{f1}/t_{f1} = 400/20\ \text{mm}$

$b_{f2}/t_{f2} = 600/40\ \text{mm}$

$h_w/t_w = 2000/8\ \text{mm}$

$b_{sl}/t_{sl} = 120/8\ \text{mm}$

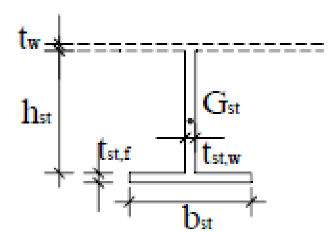
$h_{sl}/t_{sl} = 80/8\ \text{mm}$

$h_{st}/b_{st}/t_{st,w}/t_{st,f} = 180/180/10/20\ \text{mm}$

$h_{w1} = 500\ \text{mm}$

$h_{w2} = 1000\ \text{mm}$

Transverse stiffener



SELF ASSESSMENT

A two span continuous plate girder is selected in this numerical example to enable us to illustrate the design rules for the panels at the exterior and interior support as well as in the mid span. The example is not intended to show an optimal girder design but to illustrate the application of several design rules. The design loading q_{Ed} includes self-weight of the girder and all other relevant permanent and variable loads.

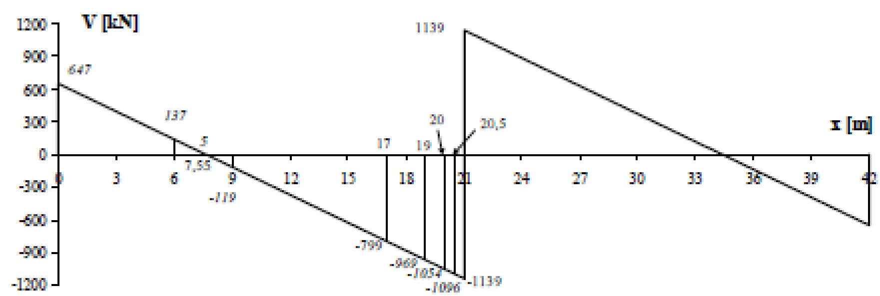


Figure 17.3: Shear force

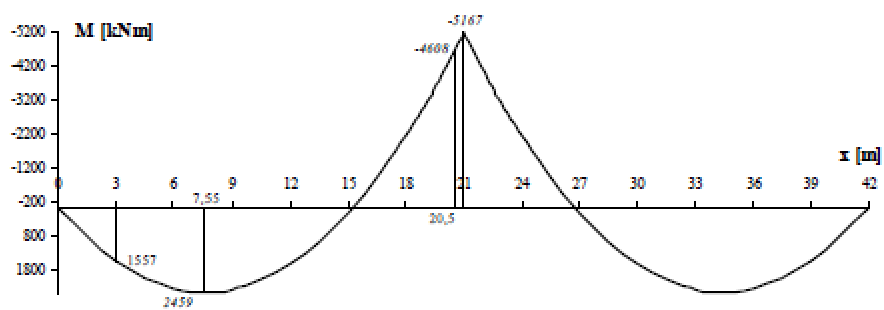


Figure 17.4: Bending moment