INTERNATIONAL UNIVERSITY OF AFRICA

CIVIL ENGINEERING DEPARTMENT

ANALYSIS AND DESIGN OF STEEL WORKS II

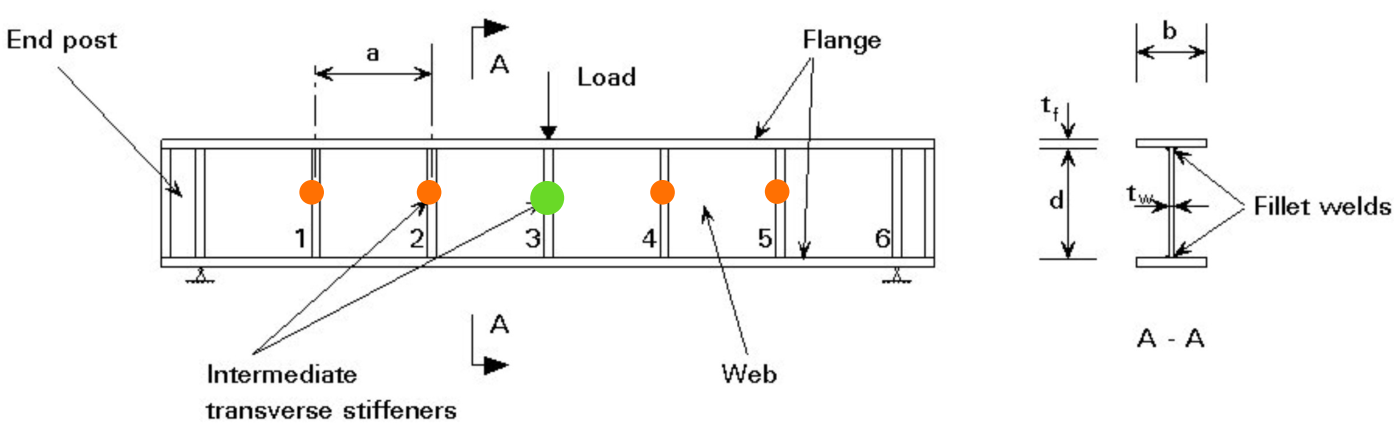
GRADE 4

8TH SEMESTER

Lecture No 7

PLATE GIRDERS

Part 2 Components



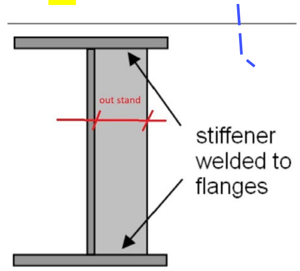
DESIGN OF OTHER COMPONENT OF THE PLATE GIRDER

I. *INTERMEDIATE STIFFENER*
II. *LOAD CARRYING STIFFENER*

I Intermediate stiffner

i. *Maximum outstand of web stiffeners outstand from the face of the web should not exceed 19εt*

ii. *. Minimum stiffness Intermediate transverse web stiffeners not subject to external loads or moments should have a second moment of area Is about the centerline of the web not less than Is given by:*



$$\text{for } a/d \geq \sqrt{2}: \qquad I_s = 0.75 d t_{min}^3$$

$$\text{for } a/d < \sqrt{2}: \qquad I_s = 1.5(d/a)^2 d t_{min}^3$$

where

a is the actual stiffener spacing;
d is the depth of the web;
t_{min} is the minimum required web thickness for the actual stiffener spacing a.

Now in our solved example

1 *Try stiffeners composed of 2 No. 60 × 8mm*
Design strength py= 275 kN/mm² *(Table 9)*
Factor ε = 1.0
Outstand 60 < 19 × 8 = 152 mm. *ok*

2 *Minimum stiffness The intermediate stiffener is shown in Figure 5. The moment of inertia about the centre of the web is*

When the spacing a = 1 000 mm<√2(1100) = 1569.5mm

a/d =1000/1100=0.9 lessthan sqrt 2

Is(provided) = $8x130^3/12=1.46x10^6$ mm^4

*Is by code =$1.5(d/a)^2 d t_{min}^3$= $1.5(1110/1000)^2$*1110*8^3*

$=1.05*10^6$

Note $t_{min} = 1200/150 = 8$ *mm*

I provided greater than I by code

$Is = 1.46x10^6 \ mm^4$ OK

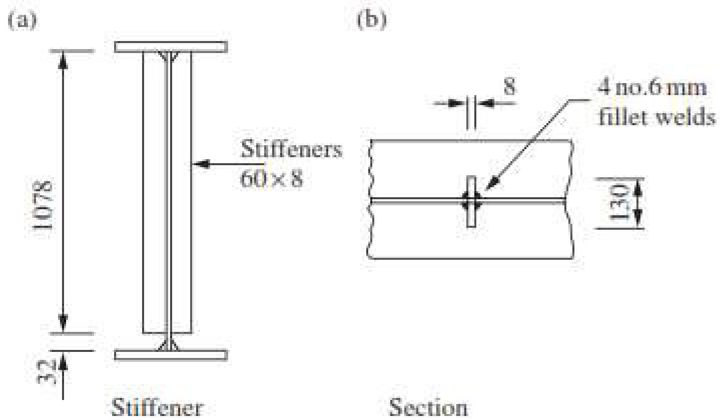


Figure 5 intermediate stiffener

3 Connection to web of intermediate stiffeners

Intermediate transverse web stiffeners that are not subject to external forces or moments should be connected to the web to withstand a shear between each component and the web (in kN per millimeter run) of not less than:

$t^2/(5b_s)$

where

b_s is the outstand of the stiffener (in mm);

t is the web thickness (in mm).

Shear generated between stiffner and the web

Shear between each flat and web = $10^2/(5x\ 60)$ = 0.333 kN/mm on two welds

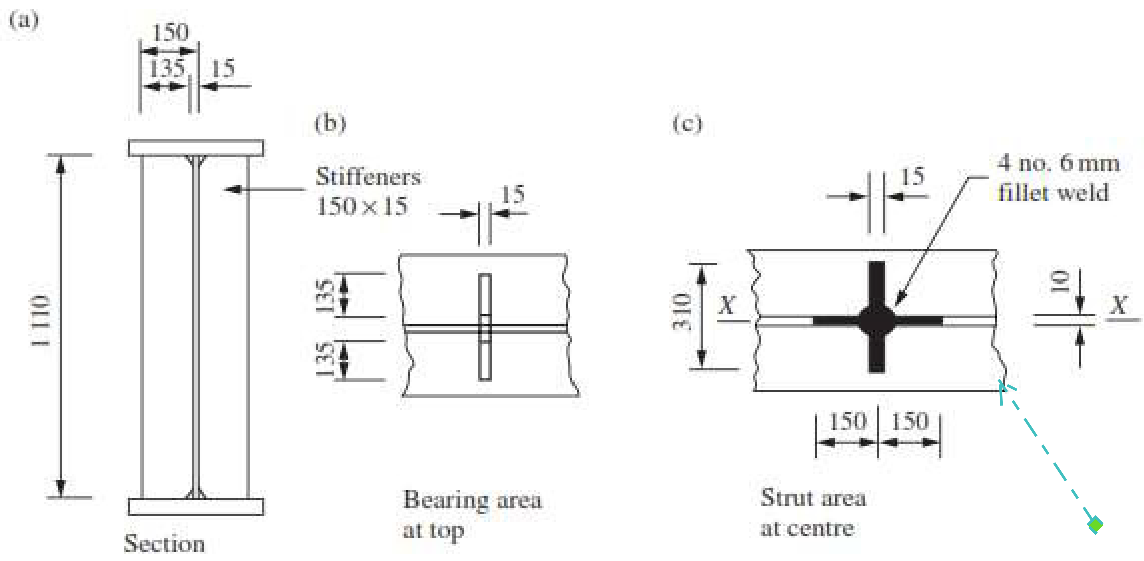
Please .refer to lecture 4 part 2 semester 7

Use 6 mm fillet weld, strength 0.924 kN/mm

Then capacity of weld providd =0.924kN/mm > 0.333 kN/mm

II. LOAD CARRYING STIFFENER

1 Try stiffeners composed of 2 No. 150×15 mm² plates as shown in Figure 6: Trial size and outstand



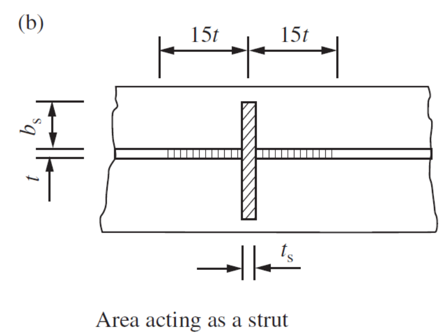


Figure 6 Load carrying and bearing stiffener

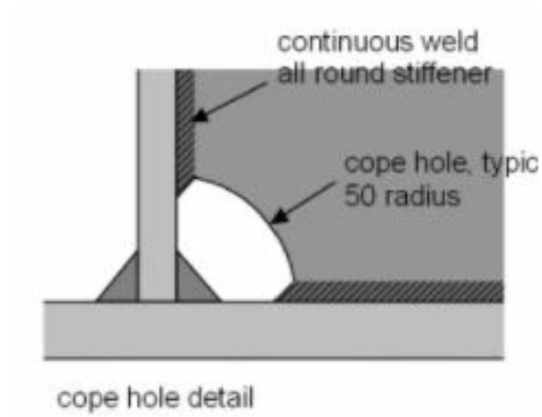
Outstand 150 < 19 × 15 = 285 mm

The stiffener is fully effective in resisting load.

(2) Bearing check

Bearing stiffeners should be minus the bearing capacity of the unstiffened web. The area of stiffener A_{snet} designed for the applied force Fx in contact with the flange is the net cross sectional area after allowing for cope holes for welding. The bearing capacityPs of the stiffener is given by:

$$P_s = A_{s.net} p_y.$$



The area in bearing at the top of the stiffener is shown in Figure 6(b). The stiffeners have been cut back 15 mm to clear the web to flange welds:

Design strength of stiffener Pys =275n/mm2 t ,16 mm table9

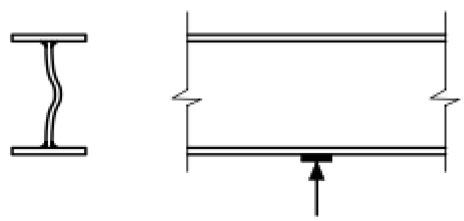
$A_{s.nets} = 2 \times 15 \times 135 = 4050\ mm^2$

$Ps = 4050 \times 275 \times 10^{-3} = = 1113.8kN > 1110\ kN$

1110kN is the load at that stiffner pls see the load configuratiin byon this link for remebering

(3) Buckling check for loading stiffeners

Load carrying stiffeners are provided to prevent the web buckling as a strut under the applied load or reaction, as shown in Figure 8. When checking for buckling, an effective web width of 15t on either side of the centerline of the stiffener is considered to act with the stiffener to form a cruciform section.



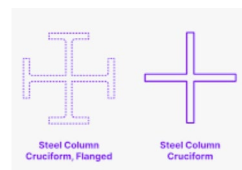


Figure 8 Buckling failure of an unstiffened web

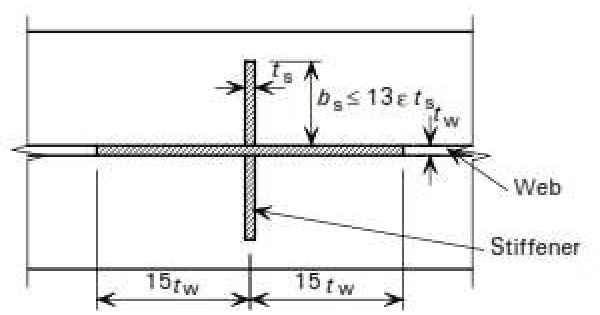


Figure 9 Effective area of load carrying stiffener

The buckling resistance of a load carrying stiffener (cruciform section as shown in Figure 9 is given by:

$$P_x = A_s p_c$$

As is the effective area, as shown in Figure 9 and includes part of the web

Pc is the compressive strength obtained from Table 24 of BS 5950-1 using strut curve c based on the design strength py and slenderness λ

λ $= L_E/r$

r is the radius of gyration of the effective area $= (Is/As)^{0.5}$

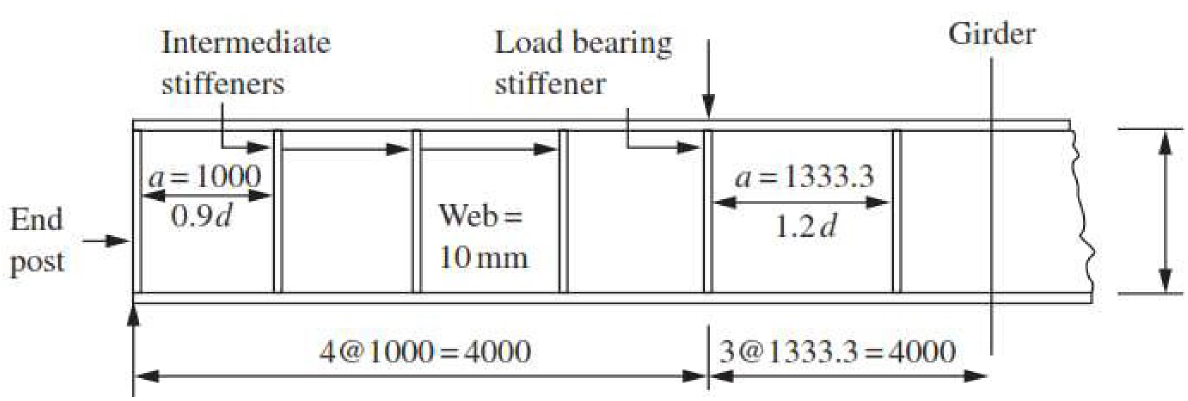
Is is the second moment of area of the effective area about the centerline of the web

Is $= t_s\,(2b_s + t_w)^3/12 + (30t_w - t_s)\,t_w^3/12.$

For cases where the flanges are restrained against relative lateral movement, the effective length L_E should be taken as:

- *If the flange is restrained against rotation in the plane of the stiffener, the effective length may be taken as 0.7d*
- *If the flange is not so restrained then the effective length should be taken as 1.0d.*

Now in our solved example



The stiffener area at the centre of the girder acting as a strut is shown in figure 6 (c)

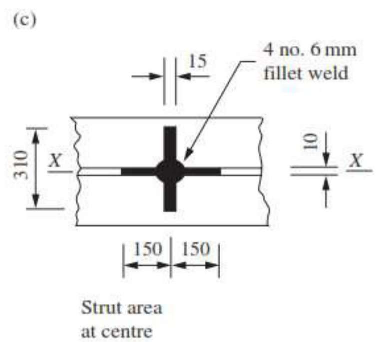


Figure 6 c

The stiffener properties are calculated from the dimensions shown:

$$A = (2 \times 150 \times 15) + (300 \times 10) = 7500\,\text{mm}^2$$
$$I_x = 15 \times 310^3/12 = 37.23 \times 10^6\,\text{mm}^4$$
$$R_x = (37.23 \times 106/7500)^{0.5} = 70.5\,\text{mm}$$

Slenderness $\lambda = 0.7 \times 1110/70.5 = 11.2$

Assume that the flange is restrained against lateral movement and against rotation in the plane of the stiffeners:

the stiffener will be designed as per lecture 6 in 7 semester compression memeber

reminder

get λ then select type curve usedtable 22 then p_c from table 24

Slenderness $\lambda = 0.7 \times 1110/70.5 = 11.2$

Design strength $= 275\,\text{N/mm}^2$

1) *select the curve type from table 23*
2 then go to table 24 both are included at the ens

Compressive strength $p_c = 275\,\text{N/mm}^2$ (Table 24 for strut curve c)

Buckling resistance:

$$P_x = 275 \times 7500/10^3 = 2062.5\,\text{kN}$$

P_X= 2062.5 >1110 kN then *OK*

The size selected is satisfactory.

4 Connection Of Load Bearing Stiffener To The Web

Where the intermediate transverse stiffener is subject to external forces, the weld connecting the stiffener to the web should be designed to carry a shear of t2/5bs kN/mm run. Plus the effect of shear due to the external load

Now in our solved example

Max bs=13Ets=13x15= 195 mm

- *Shear between each flat and web = 15^2/(5x 130) = 0.346kN/*

External l o a d mm= 1110 kN dived by 2d then dived by 2 run of weld

- *Shear due to external load =1110/(2*1110)/2* *=0.25 kN/mm*
- *Sum of shear* *0.596 kN/mm*

Use 6 mm fillet weld, strength 0.924 kN/mm
Then capacity of weld provided =0.924kN/mm > 0.583 kN/mm *ok*

8 -Connection of flange to web in welded plate girders

Aft=area of top flange.
Afb= area of bottom flange
Ix= moment of inertia about major axis
Qxt = first moment of area of top flange about neutral axis
Qxb = first moment of area- of bottom flange about neutral axis
V = ultimate shear at the section under consideration
ht= distance from neutral axis to centroid of top flange
hb = distance from neutral axis to centroid of bottom flange

$Q_{xt} = A_{ft.}*h_t$

$Q_{xb}=A_{fb}*h_b$

Vxt = horizontal shear per unit length at the connection between top flange and web

$=(V*Q_{xt})/Ix$

The capacity of two runs of fillet weld or butt weld should be greater than Vxt or Vxb as appropriate. The capacity of weld runs in kN/mm can be obtained from appropriate tables

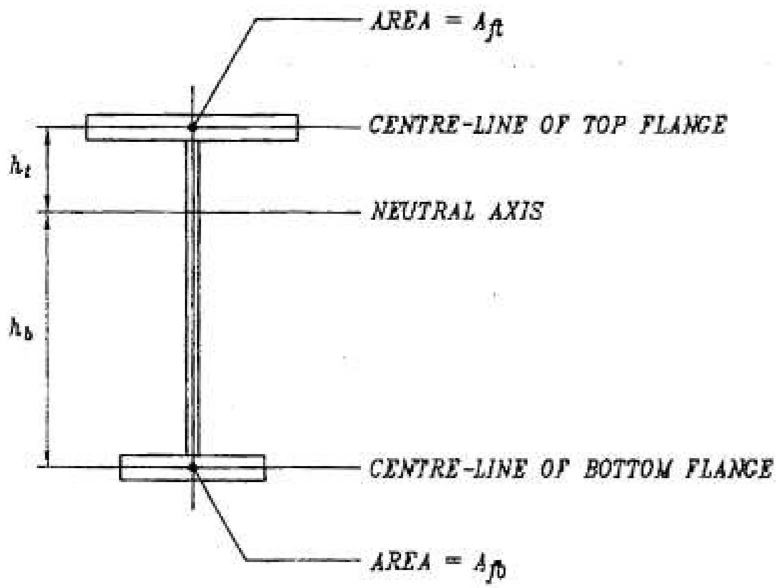
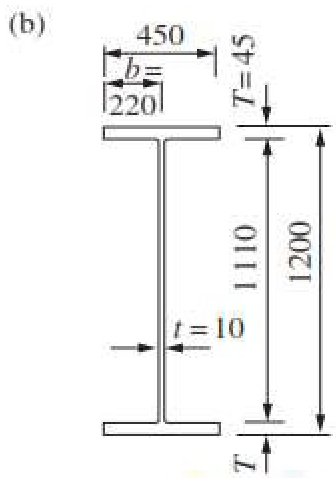


Figure 10Connection of flange to web of plate girder

Now in our solved example



Section for depth 1200 mm

Vmax = Maximum Shear =1373 kN

V Shear force on each stiffener = 1373/2 kN

Qxt= Aft ht=450*45*(1200/2-45/20 =11684 mm3*

Ix (NA at mid-point) $=10*1110^3/12+2\{450*45^3/12+(450*45(1200/2-45/2)^2\}$

$=1.463*10^{10}\ mm^4$

Horizontal shear per weld= $=(V*Qxt)/Ix \ =\{(10^{=}*1373/2)*11684\}/1.463E10)$

$=0.549\ N/mm\ run$

If we choose weld of 6 mm grade E43 electrode
the capacity of weld /mm run =0.924 N/mm run > 0.549 N/mm the applied stresses
the ok

The factored loads are:

Concentrated loads = $(1.4 \times 450) + (1.6 \times 300) = 1110\,\text{kN}$
Distributed load = $(1.4 \times 20) + (1.6 \times 10) = 44\,\text{kN/m}$

The loads and reactions are shown in Figure 2(a) and the shear force diagram in Figure 2 (b). The moments are: in Figure 2 (c)

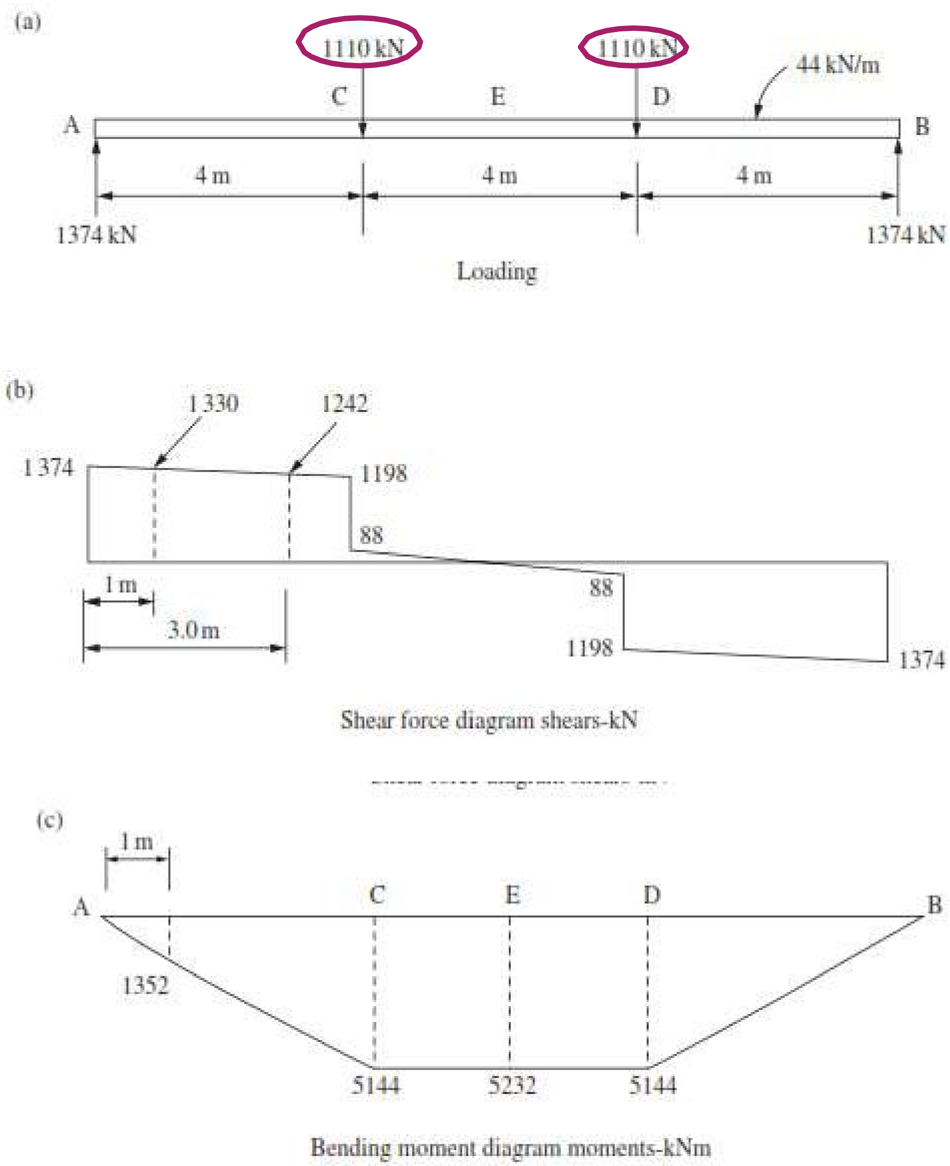


Figure 3 Load, shear and moment diagrams

Table 23 — Allocation of strut curve

Type of section	Maximum thickness (see note 1)	Axis of buckling	
		x-x	y-y
Hot-finished structural hollow section		a)	a)
Cold-formed structural hollow section		c)	c)
Rolled I-section	≤ 40 mm	a)	b)
	> 40 mm	b)	c)
Rolled H-section	≤ 40 mm	b)	c)
	> 40 mm	c)	d)
Welded I or H-section (see note 2 and 4.7.5)	≤ 40 mm	b)	c)
	> 40 mm	b)	d)
Rolled I-section with welded flange cover plates with 0.25 < U/B < 0.8 as shown in Figure 14a)	≤ 40 mm	a)	b)
	> 40 mm	b)	c)
Rolled H-section with welded flange cover plates with 0.25 < U/B < 0.8 as shown in Figure 14a)	≤ 40 mm	b)	c)
	> 40 mm	c)	d)
Rolled I or H-section with welded flange cover plates with U/B ≥ 0.8 as shown in Figure 14b)	≤ 40 mm	b)	a)
	> 40 mm	c)	b)
Rolled I or H-section with welded flange cover plates with U/B ≤ 0.25 as shown in Figure 14c)	≤ 40 mm	b)	c)
	> 40 mm	b)	d)
Welded box section (see note 3 and 4.7.5)	≤ 40 mm	b)	b)
	> 40 mm	c)	c)
Round, square or flat bar	≤ 40 mm	b)	b)
	> 40 mm	c)	c)
Rolled angle, channel or T-section Two rolled sections laced, battened or back-to-back Compound rolled sections		Any axis: c)	

NOTE 1 For thicknesses between 40 mm and 50 mm the value of p_c may be taken as the average of the values for thicknesses up to 40 mm and over 40 mm for the relevant value of p_y.

NOTE 2 For welded I or H-sections with their flanges thermally cut by machine without subsequent edge grinding or machining, for buckling about the y-y axis, strut curve b) may be used for flanges up to 40 mm thick and strut curve c) for flanges over 40 mm thick.

NOTE 3 The category "welded box section" includes any box section fabricated from plates or rolled sections, provided that all of the longitudinal welds are near the corners of the cross-section. Box sections with longitudinal stiffeners are NOT included in this category.

Table 24 — Compressive strength p_c (N/mm²) (*continued*)

5) Values of p_c (N/mm²) with λ < 110 for strut curve c														
λ	**Steel grade and design strength p_y (N/mm²)**													
	S 275					**S 355**					**S 460**			
	235	**245**	**255**	**265**	**275**	**315**	**325**	**335**	**345**	**355**	**400**	**410**	**430**	**440**
15	235	245	255	265	275	315	325	335	345	355	398	408	427	436
20	233	242	252	261	271	308	317	326	336	345	387	396	414	424
25	226	235	245	254	263	299	308	317	326	335	375	384	402	410
30	220	228	237	246	255	289	298	307	315	324	363	371	388	396
35	213	221	230	238	247	280	288	296	305	313	349	357	374	382
40	206	214	222	230	238	270	278	285	293	301	335	343	358	365
42	203	211	219	227	235	266	273	281	288	296	329	337	351	358
44	200	208	216	224	231	261	269	276	284	291	323	330	344	351
46	197	205	213	220	228	257	264	271	279	286	317	324	337	344
48	195	202	209	217	224	253	260	267	274	280	311	317	330	337