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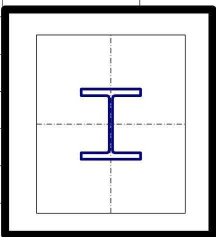
Member Design - Steel BeamColumn		Made by XX	Date 21/11/2021	Chd.

Material Properties

BS5950

Steel grade =	S275 (43)		
Design strength, p_y =	265	N/mm^2	
$\varepsilon = \sqrt{(275/p_y)}$ =	1.02		
Modulus of elasticity, E =	205000	N/mm^2	

Section



Equivalent I-Section

Section =	UC 356x406x287		
Mass per metre, m =	287.1	kg/m	
Overall utilisation =	22%		OK

Spans

Euler effective length, L_{ex} =	4.854	m	*T.22*
Euler effective length, L_{ey} =	4.740	m	*T.22*
Euler effective length, L_{ev} =	4.797	m	*T.22*
LTB effective length, $L_{E,LTB}$ =	4.797	m	*T.13, T.14*
Span, L =	3.198	m	

LTB length cannot be shortened with stiffeners, which only increases local torsional stiffness;

Loading

Elem	Case	F_x [kN]	F_y [kN]	F_z [kN]	M_{xx} [kNm]	M_{yy} [kNm]	M_{zz} [kNm]
SColumn	ULS	-1387	0	0	0	30	11

Note that for F_x, tension is positive and compression negative;

Note that these effects are effects on a particular section, e.g. mid-span section OR end support section, and thus are not necessarily max values over entire member;

Axial force, F (GSA F_x) =	1387	kN
Shear force in y-plane, V_x (GSA F_z) =	0	kN
Shear force in x-plane, V_y (GSA F_y) =	0	kN
Bending moment in y-plane, M_x (GSA M_{yy}) =	30	kNm
Bending moment in x-plane, M_y (GSA M_{zz}) =	11	kNm

Unfactored live load, ω_{LL} =	0.0	kN/m
Unfactored SLS load, ω_{SLS} =	0.0	kN/m

Deflection

Support for deflections =	Simply supported	
Percentage of dead and superimposed dead load deflection precam	0.0	%

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Additional Parameters — BS5950

Section, Orientation and Connection Type

Type of section and process =	Rolled I		
Hot finished or cold formed rolled RHS and rolled CHS =	Hot finished		
Rolled Double Angle orientation =	Both sides of support, short side connected		
Rolled Double Channel orientation =	Both sides of support, web connected		
Rolled Double T orientation =	Both sides of support, flange connected		
Bolted or welded connection type =	Welded		
No. of bolt holes (for each section if double sections), $N_{bolthole}$ =	8		
Diameter of bolt holes, $d_{bolthole}$ =	22.0	mm	

Axial Tension or Compression Capacity

Rolled I and Welded I connection connectivit	Web and both flanges connected	
Rolled RHS and Welded RHS connectior	Both webs and both flanges connected	
Rolled CHS connection connectivity =	Thickness connected	
Plate connection connectivity =	Web connected	
Rolled Single Angle connection connectivity =	Short side connected	
Rolled Single Channel connection connectivity =	Web connected	
Rolled Single T connection connectivity =	Flange connected	
Rolled Double Angle connection connectivity =	*Short side connected*	
Rolled Double Channel connection connectivity =	*Web connected*	
Rolled Double T connection connectivity =	*Flange connected*	
Rolled Double Angle connection type =	*Both sides - welded*	
Rolled Double Channel connection type =	*Both sides - welded*	
Rolled Double T connection type =	*Both sides - welded*	

Flexural Buckling Capacity

Maximum allowable slende	Members resisting loads other than wind - 180	
Robertson constant, a (option for I type section) =	Rolled H	
Rolled Single Angl	Short side connected - two bolts, standard and kidney clearance	
Rolled Single Channel connection t	Web connected - two or more rows of bolts	
Rolled Single T connection type =	Flange connected - two or more rows of bolts	
Rolled Double Angle connection type	Two or more bolts, standard clearance	
Rolled Double Angle connection type =		
ides of support, short side connected - two or more bolts, standard clearance		
Rolled Double Channel connection type =		
	Both sides of support, web connected	
Rolled Double T connection type =		
	Both sides of support, flange connected	
Rolled Double Angle, Rolled Double Channel, R(	Eleven batten(s) within span L	

LTB Capacity

Rolled Single Angle, Rolled Double Angle LTB mo	Heel of angle in tension	
Rolled Single T, Rolled Double T LTB moment ori	Flange of T in compression	
Equivalent uniform moment factor for LTB, m_{LT} =	1.000	*4.3.6.6*

Overall Buckling Capacity

Equivalent uniform moment factor for flexural buckling, m_x =	1.000	*4.8.3.3.4*
Equivalent uniform moment factor for flexural buckling, m_y =	1.000	*4.8.3.3.4*
Equivalent uniform moment factor for flexural buckling, m_{yx} =	1.000	*4.8.3.3.4*

Note that uniform moment factors are not to be used for sway sensitive structures; *4.8.3.3.4*

Utilisation Summary

BS5950

Checks	UT	Status	Overall
Cross Section Classification	36%	OK	
Shear X Capacity	0%	OK	
Shear Buckling X Capacity	0%	OK	
Shear Y Capacity	0%	OK	
Shear Buckling Y Capacity	0%	OK	
Tension or Compression Capacity	14%	OK	
Moment Capacity	2%	OK	
Local Capacity	18%	OK	22%
Slenderness Capacity	26%	OK	
Flexural Buckling Capacity	17%	OK	
Lateral Torsional Buckling Capacity	2%	OK	
Overall Buckling (Simple) Capacity	22%	OK	
Overall Buckling (Exact) Capacity	21%	OK	
Deflection (Live Load)	0%	OK	
Deflection (SLS Load)	0%	OK	

*Note the overall utilisation does not include cross section classification, shear buckling or slenderness utilisations; If the section is not **at least semi compact**, then the equations within the sheet are NOT valid! If the section classification, shear buckling or slenderness utilisations is violated, the overall utilisation is set at 999%;*

Dead and superimposed dead load deflection precamber = **0.0** mm

Beam weight = m.L (single sections) or 2m.L (double sectio **918** kg

Note web bearing and buckling capacity is not included in the above overall utilisation;

Unstiffened web bearing and buckling capacity utilisation =	**0%**	**OK**
Stiffened web bearing and buckling capacity utilisation =	**0%**	**OK**
Outstand of web stiffener length limit utilisation =	**85%**	**OK**

Typical Initial Span / Depth Ratios

Table 11:
Column size estimate based on storey of structure ***(from section 5.3 of The Institution of Structural Engineers' Manual for the design of Steel Structures to Eurocode 3)***

Number of storeys	Column size
3	203x203 UC
5	254x254 UC
8	305X305 UC
8-12	356X356 UC

Table 10: Span/depth ratio tables for steel beams located in the floor a

Type of beam	Maximum floor span	Depth of
Primary beams	15m	Sp
Secondary beams	12m	Sp

Tata Steel Europe website)

Maximum roof span	Depth of roof beam
15m	Span/25
15m	Span/25

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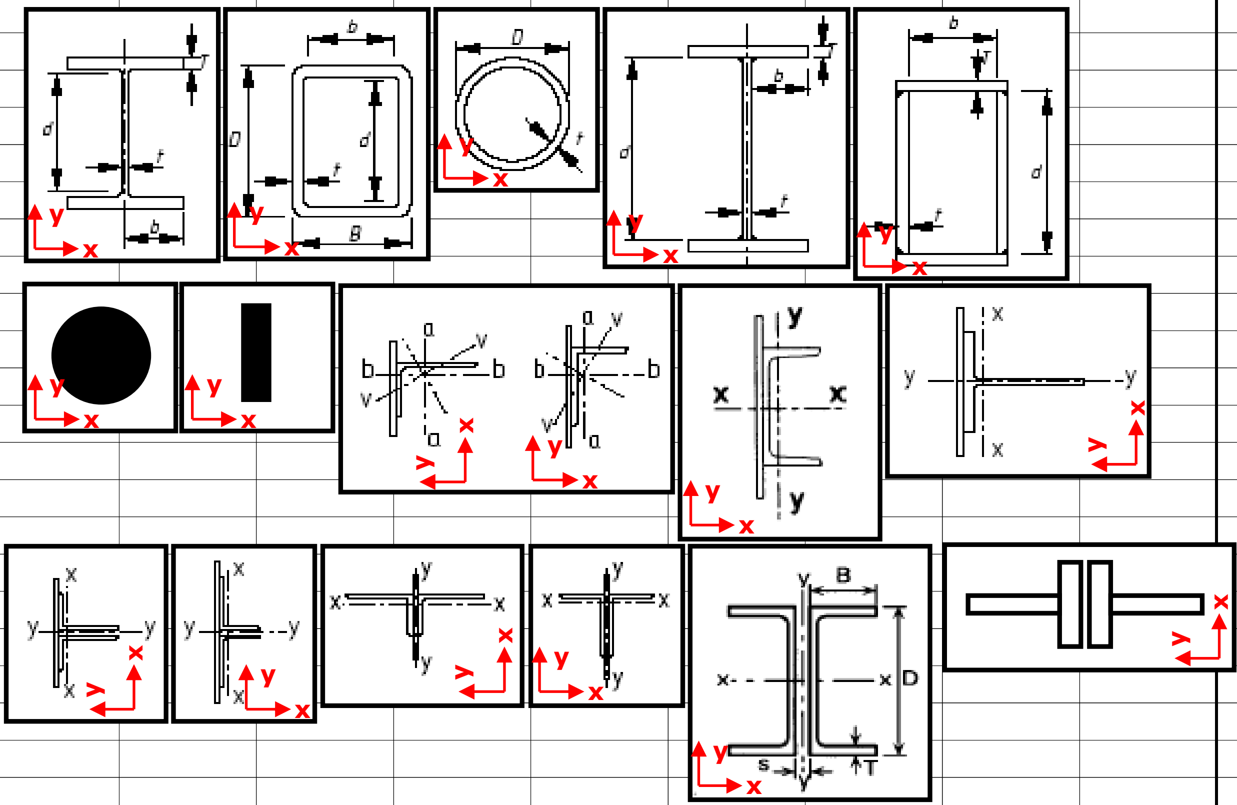
Axes Definition BS5950



Note where relevant, it is assumed that double sections bend compositely, i.e. that in theory there is full shear connectivity between the sections. The equations contained within the sheet reflect this assumption;

nd roof (from T
floor beam
an/20
an/25

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General Section Properties			*BS5950*
Total depth, D =	**393.6**	mm	
Web thickness, t =	**22.6**	mm	
Flange width, B =	**399.0**	mm	
Flange thickness, T =	**36.5**	mm	
Root radius, r_i =	**15.2**	mm	
Rolled I b/T ratio =	**5.5**		
Rolled I d/t ratio =	**12.8**		
Gross area of section, A_g =	**366.0**	cm^2	
Second moment of area about x-x axis, I_x =	**99875**	cm^4	
Second moment of area about y-y axis, I_y =	**38677**	cm^4	
Second moment of area about major axis, I_u =	**N/A**	cm^4	
Second moment of area about minor axis, I_v =	**N/A**	cm^4	
Radius of gyration about x-x axis, r_x =	**16.5**	cm	
Radius of gyration about y-y axis, r_y =	**10.3**	cm	
Radius of gyration about major axis, r_u =	**N/A**	cm	
Radius of gyration about minor axis, r_v =	**N/A**	cm	
Elastic modulus about x-x axis, Z_x =	**5075**	cm^3	
Elastic modulus about y-y axis, Z_y =	**1939**	cm^3	
Plastic modulus about x-x axis, s_x =	**5812**	cm^3	
Plastic modulus about y-y axis, s_y =	**2949**	cm^3	
Buckling parameter, u =	**0.835**		
Torsional index, x =	**10.2**		
Torsional constant, J =	**1441**	cm^4	
Warping constant, H =	**12.3**	dm^6	
Monosymmetric index, ψ =	**N/A**		
K_e = (1.2 for G43, 1.1 for G50, 1.0 for G55) =	**1.2**		*3.4.2*

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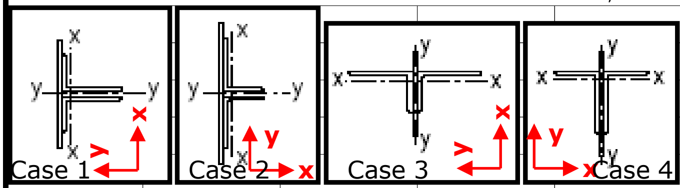
Particular Section Properties

BS5950

Particular Section Properties for Rolled Double Angle

Gross area of section, $A_{g, double\ angle}$ = $2A_g$ =	**N/A**	cm^2
Space between sections, s =	15.0	mm
Centroid of section from back face to x-x axis, $x_{c, double\ angle, x}$ = (B.T	N/A	mm
Centroid of section from back face to y-y axis, $x_{c, double\ angle, y}$ = (D.t	N/A	mm



Applicable case = **N/A**

	$I_{x, double\ angle}$ (cm^4)	$I_{y, double\ angle}$ (cm^4)	$Z_{x, double\ angle}$ (cm^3)	$Z_{y, double\ angle}$ (cm^3)
Case 1	N/A	N/A	N/A	N/A
Case 2	N/A	N/A	N/A	N/A
Case 3	N/A	N/A	N/A	N/A
Case 4	N/A	N/A	N/A	N/A

Second moment of area about x-x axis, $I_{x, double\ angle}$ =	**N/A**	cm^4
Second moment of area about y-y axis, $I_{y, double\ angle}$ =	**N/A**	cm^4
Radius of gyration about x-x axis, $r_{x, double\ angle}$ = √($I_{x, double\ an}$	**N/A**	cm
Radius of gyration about y-y axis, $r_{y, double\ angle}$ = √($I_{y, double\ an}$	**N/A**	cm
Elastic modulus about x-x axis, $Z_{x, double\ angle}$ =	**N/A**	cm^3
Elastic modulus about y-y axis, $Z_{y, double\ angle}$ =	**N/A**	cm^3
Plastic modulus about x-x axis, $s_{x, double\ angle}$ = N/A =	**N/A**	cm^3
Plastic modulus about y-y axis, $s_{y, double\ angle}$ = N/A =	**N/A**	cm^3

Particular Section Properties for Rolled Double Channel

Gross area of section, $A_{g, double\ channel}$ = $2A_g$ =	**N/A**	cm^2
Space between sections, s =	270.0	mm
Centroid of section from back face, $x_{c, double\ channel}$ = (B.T.(B/2).2+(I	N/A	mm
Second moment of area about x-x axis, $I_{x, double\ channel}$ = $2I_x$ =	**N/A**	cm^4
Second moment of area about y-y axis, $I_{y, double\ channel}$ = 2.(I_y	**N/A**	cm^4
Radius of gyration about x-x axis, $r_{x, double\ channel}$ = √($I_{x, double\ c}$	**N/A**	cm
Radius of gyration about y-y axis, $r_{y, double\ channel}$ = √($I_{y, double\ c}$	**N/A**	cm
Elastic modulus about x-x axis, $Z_{x, double\ channel}$ = $2Z_x$ =	**N/A**	cm^3
Elastic modulus about y-y axis, $Z_{y, double\ channel}$ = $I_{y, double\ channel}$	**N/A**	cm^3
Plastic modulus about x-x axis, $s_{x, double\ channel}$ = $2s_x$ =	**N/A**	cm^3
Plastic modulus about y-y axis, $s_{y, double\ channel}$ = $2A_g$.(s/2+x_c,	**N/A**	cm^3

Particular Section Properties for Rolled Double T

Gross area of section, $A_{g, double\ T}$ = $2A_g$ =	**N/A**	cm^2
Space between sections, s =	15.0	mm
Centroid of section from back face, $x_{c, double\ T}$ = (B.T.(T/2)+(D–T).t.	N/A	mm
Second moment of area about x-x axis, $I_{x, double\ T}$ = 2.(I_x+A_g.	**N/A**	cm^4
Second moment of area about y-y axis, $I_{y, double\ T}$ = $2I_y$ =	**N/A**	cm^4
Radius of gyration about x-x axis, $r_{x, double\ T}$ = √($I_{x, double\ T}$ / A	**N/A**	cm
Radius of gyration about y-y axis, $r_{y, double\ T}$ = √($I_{y, double\ T}$ / A	**N/A**	cm
Elastic modulus about x-x axis, $Z_{x, double\ T}$ = $I_{x, double\ T}$ / (s/2+	**N/A**	cm^3
Elastic modulus about y-y axis, $Z_{y, double\ T}$ = $2Z_y$ =	**N/A**	cm^3
Plastic modulus about x-x axis, $s_{x, double\ T}$ = $2A_g$.(s/2+$x_{c, double}$	**N/A**	cm^3
Plastic modulus about y-y axis, $s_{y, double\ T}$ = $2s_y$ =	**N/A**	cm^3

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Relevant Section Properties *BS5950*

Section Type	$A_{g, relevant}$	$I_{x, relevant}$	$I_{y, relevant}$	$I_{v, relevant}$	$r_{y, relevant}$
Rolled I	A_g	I_x	I_y	N/A	r_y
Rolled RHS	A_g	I_x	I_y	N/A	r_y
Rolled CHS	A_g	I_x	I_y	N/A	r_y
Welded I	A_g	I_x	I_y	N/A	r_y
Welded RHS	A_g	I_x	I_y	N/A	r_y
Solid Bar	A_g	I_x	I_y	N/A	r_y
Plate	A_g	I_x	I_y	N/A	r_y
Rolled Single Angle	A_g	I_x	I_y	I_v	r_y
Rolled Single Channel	A_g	I_x	I_y	N/A	r_y
Rolled Single T	A_g	I_x	I_y	N/A	r_y
Rolled Double Angle	$A_{g, double\ angle}$	$I_{x, double\ angle}$	$I_{y, double\ angle}$	N/A	$r_{y, double\ angle}$
Rolled Double Channe	$A_{g, double\ chann}$	$_{x, double\ channe}$	$_{y, double\ channe}$	N/A	$_{y, double\ channel}$
Rolled Double T	$A_{g, double\ T}$	$I_{x, double\ T}$	$I_{y, double\ T}$	N/A	$r_{y, double\ T}$

Relevant gross area of section, $A_{g, relevant}$ =	**366.0**	cm^2
Relevant second moment of area about x-x axis, $I_{x, relevant}$ =	**99875**	cm^4
Relevant second moment of area about y-y axis, $I_{y, relevant}$ =	**38677**	cm^4
Relevant second moment of area about minor axis, $I_{v, relevant}$	**N/A**	cm^4
Radius of gyration about y-y axis, $r_{y, relevant}$ =	**10.3**	cm

Section Type	$Z_{x, relevant}$	$Z_{y, relevant}$	$s_{x, relevant}$	$s_{y, relevant}$
Rolled I	Z_x	Z_y	s_x	s_y
Rolled RHS	Z_x	Z_y	s_x	s_y
Rolled CHS	Z_x	Z_y	s_x	s_y
Welded I	Z_x	Z_y	s_x	s_y
Welded RHS	Z_x	Z_y	s_x	s_y
Solid Bar	Z_x	Z_y	s_x	s_y
Plate	Z_x	Z_y	s_x	s_y
Rolled Single Angle	Z_x	Z_y	s_x	s_y
Rolled Single Channel	Z_x	Z_y	s_x	s_y
Rolled Single T	Z_x	Z_y	s_x	s_y
Rolled Double Angle	$Z_{x, double\ angle}$	$Z_{y, double\ angle}$	$s_{x, double\ angle}$	$s_{y, double\ angle}$
Rolled Double Channe	$Z_{x, double\ chann}$	$_{y, double\ chann}$	$_{x, double\ chann}$	$_{y, double\ channel}$
Rolled Double T	$Z_{x, double\ T}$	$Z_{y, double\ T}$	$s_{x, double\ T}$	$s_{y, double\ T}$

Relevant elastic modulus about x-x axis, $Z_{x, relevant}$ =	**5075**	cm^3
Relevant elastic modulus about y-y axis, $Z_{y, relevant}$ =	**1939**	cm^3
Relevant plastic modulus about x-x axis, $s_{x, relevant}$ =	**5812**	cm^3
Relevant plastic modulus about y-y axis, $s_{y, relevant}$ =	**2949**	cm^3

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Cross Section Classification (Local Buckling Effects)							BS5950
(At Least Semi Compact Section Required; At Most Compact Section;)							
y-plane	**Section classification =**			**Compact**			
x-plane	**Section classification =**			**N/A**			
y-plane	**Semi compact section classification utilisation =**				**0.358**		**OK**
x-plane	**Semi compact section classification utilisation =**				**N/A**		**N/A**
overall	**Semi compact section classification utilisation =**				**0.358**		**OK**
Compact Section Classification		**Limit b/T**		**Limit d/t**			
	Rolled I [(B/2)/T,(D-2T-2r$_i$)/t]	10ε =	10.2	ε, 100ε/(1+	46.4		*3.5*
y-plane	Rolled RHS [(B-3or5t)/	32ε or 28ε, (62ε or 54	N/A	or 35ε, (80	N/A		*3.5*
x-plane	Rolled RHS	MAX(40ε or 35ε, (80	N/A	(62ε or 54	N/A		*3.5*
	Rolled CHS [D/t,D/t]	0, 50ε^2 =	N/A	0, 50ε^2 =	N/A		*3.5*
	Welded I [((B-t)/2)/T,(D-2T)/t]	9ε =	N/A	ε, 100ε/(1+	N/A		*3.5*
y-plane	Welded RHS [((B-t)/2)/T,(D-2T)/t]	32ε, 62ε-0.5d/	N/A	(40ε, 80ε/(1	N/A		*3.5*
x-plane	Welded RHS [((B-t)/2)/T,(D-2T)/t]	80ε, MAX(40ε, 80ε/(1	N/A	, 62ε-0.5b/	N/A		*3.5*
	Solid Bar [N/A,N/A]	N/A	N/A	N/A	N/A		*3.5*
	Plate [N/A,D/t]	N/A	N/A	10ε =	N/A		*3.5*
	Rolled Single Angle [B/T,D/t]	10ε =	N/A	10ε =	N/A		*3.5*
	Rolled Single Channel [B/T,(D-2T	10ε =	N/A	40ε =	N/A		*3.5*
	Rolled Single T [(B/2)/T,D/t]	10ε =	N/A	9ε =	N/A		*3.5*
	Rolled Double Angle [B/T,D/t]	10ε =	N/A	10ε =	N/A		*3.5*
	Rolled Double Channel [B/T,(D-2	10ε =	N/A	40ε =	N/A		*3.5*
	Rolled Double T [(B/2)/T,D/t]	10ε =	N/A	9ε =	N/A		*3.5*
y-plane	**Rolled I b/T ratio =**		**5.5**	<	**10.2**		**OK**
x-plane	**Rolled I b/T ratio =**		**5.5**	<	**N/A**		**N/A**
y-plane	**Rolled I d/t ratio =**		**12.8**	<	**46.4**		**OK**
x-plane	**Rolled I d/t ratio =**		**12.8**	<	**N/A**		**N/A**
	Note if 1+r$_1$ < 0.1, then 1+r$_1$ set to 0.1 to avoid singularity;						
Semi Compact Section Classification		**Limit b/T**		**Limit d/t**			
	Rolled I [(B/2)/T,(D-2T-2r$_i$)/t]	15ε =	15.3	X(40ε, 120ε	95.1		*3.5*
y-plane	Rolled RHS [(B-3or5t)/T,(D-3or5	40ε or 35ε =	N/A	ε or 35ε, (1	N/A		*3.5*
x-plane	Rolled RHS	MAX(40ε or 35ε, (1	N/A	40ε or 35ε =	N/A		*3.5*
	Rolled CHS [D/t,D/t]	80ε^2, 140ε^2	N/A	0ε^2, 140ε^2	N/A		*3.5*
	Welded I [((B-t)/2)/T,(D-2T)/t]	13ε =	N/A	X(40ε, 120ε	N/A		*3.5*
y-plane	Welded RHS [((B-t)/2)/T,(D-2T)/	40ε =	N/A	X(40ε, 120ε	N/A		*3.5*
x-plane	Welded RHS [((B-t)/2)/T,(D-2T)/t]	120ε, MAX(40ε, 120ε	N/A	40ε =	N/A		*3.5*
	Solid Bar [N/A,N/A]	N/A	N/A	N/A	N/A		*3.5*
	Plate [N/A,D/t]	N/A	N/A	15ε =	N/A		*3.5*
	Rolled Single Angle [B/T,D/t]	15ε =	N/A	15ε =	N/A		*3.5*
	Rolled Single Channel [B/T,(D-2T	15ε =	N/A	40ε =	N/A		*3.5*
	Rolled Single T [(B/2)/T,D/t]	15ε =	N/A	18ε =	N/A		*3.5*
	Rolled Double Angle [B/T,D/t]	15ε =	N/A	15ε =	N/A		*3.5*
	Rolled Double Channel [B/T,(D-2	15ε =	N/A	40ε =	N/A		*3.5*
	Rolled Double T [(B/2)/T,D/t]	15ε =	N/A	18ε =	N/A		*3.5*
y-plane	**Rolled I b/T ratio =**		**5.5**	<	**15.3**		**OK**
x-plane	**Rolled I b/T ratio =**		**5.5**	<	**N/A**		**N/A**
y-plane	**Rolled I d/t ratio =**		**12.8**	<	**95.1**		**OK**
x-plane	**Rolled I d/t ratio =**		**12.8**	<	**N/A**		**N/A**
	Note if 1+2r$_2$ < 0.1, then 1+2r$_2$ set to 0.1 to avoid singularity;						

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Member Design - Steel BeamColumn		Made by XX	Date 21/11/2021	Chd.

Shear Capacity — BS5950

Note that the shear capacity in a particular direction is independent of whether there is shear fixity in the particular direction or not, as if the latter, then shear connection capacity is provided by web bearing and buckling;

Thickness of section at hole, $t_{bolthole} = t$ =	22.6	mm	
Area of section to deduct, $A_{deduct} = t_{bolthole}.d_{bolthole}.N_{bolthole}$ =	0	mm^2	
Note A_{deduct} = 0.0 if welded connection type;			
Note A_{deduct} is doubled to $2.A_{deduct}$ for double sections;			
Rolled I shear X area, $A_{vx} = tD$ =	8895	mm^2	*4.2.3*
Rolled RHS shear X area, $A_{vx} = A_g.D/(D+B)$ =	N/A	mm^2	*4.2.3*
Rolled CHS shear X area, $A_{vx} = 0.6A_g$ =	N/A	mm^2	*4.2.3*
Welded I shear X area, $A_{vx} = t(D-2T)$ =	N/A	mm^2	*4.2.3*
Welded RHS shear X area, $A_{vx} = 2t(D-2T)$ =	N/A	mm^2	*4.2.3*
Solid Bar shear X area, $A_{vx} = 0.9A_g$ =	N/A	mm^2	*4.2.3*
Plate shear X area, $A_{vx} = 0.9A_g$ =	N/A	mm^2	*4.2.3*
Rolled Single Angle shear X area, $A_{vx} = 0.9tD$ =	N/A	mm^2	*4.2.3*
Rolled Single Channel shear X area, $A_{vx} = tD$ =	N/A	mm^2	*4.2.3*
Rolled Single T shear X area, $A_{vx} = tD$ =	N/A	mm^2	*4.2.3*
Rolled Double Angle shear X area, $A_{vx} = 2(0.9tD)$ =	N/A	mm^2	*4.2.3*
Rolled Double Channel shear X area, $A_{vx} = 2.tD$ =	N/A	mm^2	*4.2.3*
Rolled Double T shear X area, $A_{vx} = 2.tD$ =	N/A	mm^2	*4.2.3*
Shear X area, A_{vx} =	8895	mm^2	
Net shear X area, $A_{vx,net} = A_{vx} - A_{deduct}$ =	8895	mm^2	*6.2.3*

Bolt holes need not be allowed for in the shear area provided that:
$A_{v.net} \geq 0.85A_v/K_e$

Shear X area limit for significance of bolt holes, $0.85A_{vx}/K_e$ =	6301	mm^2	*6.2.3*
Shear capacity, P_{vx}	**1414**	kN	
insignificant bolt holes $P_{vx} = 0.6p_y.A_{vx}$ =	1414	kN	*4.2.3*
significant bolt holes $P_{vx} = 0.7p_y.K_e.A_{vx,net}$ =	1980	kN	*6.2.3*
V_x / P_{vx} utilisation =	**0.000**		**OK**
Shear X buckling, d/t ratio (< 70ε if rolled and 62ε if welded	**12.8 < 71.3**		*4.2.3*
Shear X buckling utilisation =	**0.000**		**OK**

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Member Design - Steel BeamColumn		Made by **XX**	Date **21/11/2021**	Chd.

			<u>*BS5950*</u>
Thickness of section at hole, $t_{bolthole}$ = T =	36.5	mm	
Area of section to deduct, $A_{deduct} = t_{bolthole}.d_{bolthole}.N_{bolthole}$ =	0	mm^2	
Note A_{deduct} = 0.0 if welded connection type;			
Note A_{deduct} is doubled to 2.A_{deduct} for double sections;			
Rolled I shear Y area, A_{vy} = 0.9(2BT) =	26214	mm^2	*4.2.3*
Rolled RHS shear Y area, A_{vy} = 0.9(2BT) =	N/A	mm^2	*4.2.3*
Rolled CHS shear Y area, $A_{vy} = 0.6A_g$ =	N/A	mm^2	*4.2.3*
Welded I shear Y area, A_{vy} = 0.9(2BT) =	N/A	mm^2	*4.2.3*
Welded RHS shear Y area, A_{vy} = 0.9(2BT) =	N/A	mm^2	*4.2.3*
Solid Bar shear Y area, $A_{vy} = 0.9A_g$ =	N/A	mm^2	*4.2.3*
Plate shear Y area, $A_{vy} = 0.9A_g$ =	N/A	mm^2	*4.2.3*
Rolled Single Angle shear Y area, A_{vy} = 0.9BT =	N/A	mm^2	*4.2.3*
Rolled Single Channel shear Y area, A_{vy} = 0.9(2BT) =	N/A	mm^2	*4.2.3*
Rolled Single T shear Y area, A_{vy} = 0.9BT =	N/A	mm^2	*4.2.3*
Rolled Double Angle shear Y area, A_{vy} = 2(0.9BT) =	N/A	mm^2	*4.2.3*
Rolled Double Channel shear Y area, A_{vy} = 2(0.9(2BT)) =	N/A	mm^2	*4.2.3*
Rolled Double T shear Y area, A_{vy} = 2(0.9BT) =	N/A	mm^2	*4.2.3*
Shear Y area, A_{vy} =	26214	mm^2	
Net shear Y area, $A_{vy,net} = A_{vy} - A_{deduct}$ =	26214	mm^2	*6.2.3*

Bolt holes need not be allowed for in the shear area provided that:

$A_{v.net} \geq 0.85A_v/K_e$

Shear Y area limit for significance of bolt holes, $0.85A_{vy}/K_e$ =	18568	mm^2	*6.2.3*
Shear capacity, P_{vy}	**4168**	kN	
insignificant bolt holes $P_{vy} = 0.6p_y.A_{vy}$ =	4168	kN	*4.2.3*
significant bolt holes $P_{vy} = 0.7p_y.K_e.A_{vy,net}$ =	5835	kN	*6.2.3*
V_y / P_{vy} utilisation =	**0.000**		**OK**
Shear Y buckling, b/T ratio (< 70ε if rolled and 62ε if welded	**5.5 < 71.3**		*4.2.3*
Shear Y buckling utilisation =	**0.000**		**OK**

Axial Tension or Compression Capacity — *BS5950*

Note that A_e is applicable for axial tension with bolted connections;
Note that A_g is applicable for axial tension with welded connections;
Note that A_g (as A_e becomes A_g) is applicable for axial compression with bolted connections;
Note that A_g is applicable for axial compression with welded connections;

> Tests show that holes do not reduce the capacity of a member in tension provided that the ratio of net area to gross area is greater than the ratio of yield strength to ultimate strength.

Rolled I and Welded I

Gross area of connected part(s), a_1 = (D.t, 2B.T, D.t+B.T or A_g) =	366.0	cm^2	
Gross area of unconnected part(s), $a_2 = A_g - a_1$ =	0.0	cm^2	
Thickness of section at hole, $t_{bolthole,I}$ =	36.5	mm	
Area of section to deduct, $A_{deduct,I} = t_{bolthole,I}.d_{bolthole}.N_{bolthole}$ =	0.0	cm^2	
Note $A_{deduct,I}$ = 0.0 if welded connection type;			
Net area of connected part(s), $a_{n1} = a_1 - A_{deduct,I}$ =	366.0	cm^2	
Net area of unconnected part(s), $a_{n2} = a_2$ =	0.0	cm^2	
Effective net area of connected part(s), $a_{e1} = K_e.a_{n1}$ (<= a_1) =	366.0	cm^2	
Effective net area of unconnected part(s), $a_{e2} = K_e.a_{n2}$ (<= a_2) =	0.0	cm^2	
Effective net area of section, $A_{e,I}$ = [If F_x > 0, $a_{e1}+a_{e2}$ (<= 1.	**366.0**	cm^2	
Bolted P_t or $P_{ca} = p_y(A_{e,I}-1.00a_2)$ =	9699	kN	*4.6.1*
Welded P_t or $P_{ca} = p_y(A_g-1.00a_2)$ =	9699	kN	*4.6.1*
Axial tension or compression capacity, P_t or P_{ca} =	**9699**	kN	

Rolled RHS and Welded RHS

Gross area of connected part(s), a_1 = (2D.t, 2B.T, 2D.t+B.T or A_g)	N/A	cm^2	
Gross area of unconnected part(s), $a_2 = A_g - a_1$ =	N/A	cm^2	
Thickness of section at hole, $t_{bolthole,RHS}$ =	N/A	mm	
Area of section to deduct, $A_{deduct,RHS} = t_{bolthole,RHS}.d_{bolthole}.N_{bolthole}$ =	N/A	cm^2	
Note $A_{deduct,RHS}$ = 0.0 if welded connection type;			
Net area of connected part(s), $a_{n1} = a_1 - A_{deduct,I}$ =	N/A	cm^2	
Net area of unconnected part(s), $a_{n2} = a_2$ =	N/A	cm^2	
Effective net area of connected part(s), $a_{e1} = K_e.a_{n1}$ (<= a_1) =	N/A	cm^2	
Effective net area of unconnected part(s), $a_{e2} = K_e.a_{n2}$ (<= a_2) =	N/A	cm^2	
Effective net area of section, $A_{e,RHS}$ = [If F_x > 0, $a_{e1}+a_{e2}$ (<=	**N/A**	cm^2	
Bolted P_t or $P_{ca} = p_y(A_{e,RHS}-1.00a_2)$ =	N/A	kN	*4.6.1*
Welded P_t or $P_{ca} = p_y(A_g-1.00a_2)$ =	N/A	kN	*4.6.1*
Axial tension or compression capacity, P_t or P_{ca} =	**N/A**	kN	

Rolled CHS

Thickness of section at hole, $t_{bolthole,CHS}$ =	N/A	mm	
Area of section to deduct, $A_{deduct,CHS} = t_{bolthole,CHS}.d_{bolthole}.N_{bolthole}$ =	N/A	cm^2	
Note $A_{deduct,CHS}$ = 0.0 if welded connection type;			
Net area of section, $A_{net,CHS} = A_g - A_{deduct,CHS}$ =	N/A	cm^2	
Effective net area of section, $A_{e,CHS}$ = [If F_x > 0, MIN ($K_e.A_{net,}$	**N/A**	cm^2	
Bolted P_t or $P_{ca} = p_yA_{e,CHS}$ =	N/A	kN	*4.6.1*
Welded P_t or $P_{ca} = p_yA_g$ =	N/A	kN	*4.6.1*
Axial tension or compression capacity, P_t or P_{ca} =	**N/A**	kN	

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		Member/Location		
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Member Design - Steel BeamColumn		Made by **XX**	Date **21/11/2021**	Chd.

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Solid Bar				
Effective net area of section, $A_{e,bar}$ = A_g =		**N/A**	cm^2	
Bolted	P_t or P_{ca} = $p_y A_{e,bar}$ =	N/A	kN	*4.6.1*
Welded	P_t or P_{ca} = $p_y A_g$ =	N/A	kN	*4.6.1*
Axial tension or compression capacity, P_t or P_{ca} =		**N/A**	kN	
Plate				
Thickness of section at hole, $t_{bolthole,plate}$ =		N/A	mm	
Area of section to deduct, $A_{deduct,plate}$ = $t_{bolthole,plate}.d_{bolthole}.N_{bolthole}$ =		N/A	cm^2	
Note $A_{deduct,plate}$ = 0.0 if welded connection type;				
Net area of section, $A_{net,plate}$ = A_g – $A_{deduct,plate}$ =		N/A	cm^2	
Effective net area of section, $A_{e,plate}$ = [If F_x > 0, MIN ($K_e.A_{ne}$		**N/A**	cm^2	
Bolted	P_t or P_{ca} = $p_y A_{e,plate}$ =	N/A	kN	*4.6.1*
Welded	P_t or P_{ca} = $p_y A_g$ =	N/A	kN	*4.6.1*
Axial tension or compression capacity, P_t or P_{ca} =		**N/A**	kN	
Rolled Single Angle				
Gross area of connected leg, a_1 = (B.T or D.t) =		N/A	cm^2	
Gross area of unconnected leg, a_2 = A_g - a_1 =		N/A	cm^2	
Thickness of section at hole, $t_{bolthole,single\ angle}$ =		N/A	mm	
Area of section to deduct, $A_{deduct,single\ angle}$ = $t_{bolthole,single\ angle}.d_{bolthole}.N$		N/A	cm^2	
Note $A_{deduct,single\ angle}$ = 0.0 if welded connection type;				
Net area of connected leg, a_{n1} = a_1 - $A_{deduct,single\ angle}$ =		N/A	cm^2	
Net area of unconnected leg, a_{n2} = a_2 =		N/A	cm^2	
Effective net area of connected leg, a_{e1} = $K_e.a_{n1}$ (<= a_1) =		N/A	cm^2	
Effective net area of unconnected leg, a_{e2} = $K_e.a_{n2}$ (<= a_2) =		N/A	cm^2	
Effective net area of section, $A_{e,single\ angle}$ = [If F_x > 0, a_{e1}+a_{e2}		**N/A**	cm^2	
Bolted	P_t or P_{ca} = $p_y(A_{e,single\ angle}$-0.50a_2) =	N/A	kN	*4.6.3.1*
Welded	P_t or P_{ca} = $p_y(A_g$-0.30a_2) =	N/A	kN	*4.6.3.1*
Axial tension or compression capacity, P_t or P_{ca} =		**N/A**	kN	
Rolled Single Channel				
Gross area of connected leg, a_1 = D.t =		N/A	cm^2	
Gross area of unconnected leg, a_2 = A_g - a_1 =		N/A	cm^2	
Thickness of section at hole, $t_{bolthole,single\ channel}$ =		N/A	mm	
Area of section to deduct, $A_{deduct,single\ channel}$ = $t_{bolthole,single\ channel}.d_{boltho}$		N/A	cm^2	
Note $A_{deduct,single\ channel}$ = 0.0 if welded connection type;				
Net area of connected leg, a_{n1} = a_1 - $A_{deduct,single\ channel}$ =		N/A	cm^2	
Net area of unconnected leg, a_{n2} = a_2 =		N/A	cm^2	
Effective net area of connected leg, a_{e1} = $K_e.a_{n1}$ (<= a_1) =		N/A	cm^2	
Effective net area of unconnected leg, a_{e2} = $K_e.a_{n2}$ (<= a_2) =		N/A	cm^2	
Effective net area of section, $A_{e,single\ channel}$ = [If F_x > 0, a_{e1}+a		**N/A**	cm^2	
Bolted	P_t or P_{ca} = $p_y(A_{e,single\ channel}-0.50a_2$) =	N/A	kN	*4.6.3.1*
Welded	P_t or P_{ca} = $p_y(A_g$-0.30a_2) =	N/A	kN	*4.6.3.1*
Axial tension or compression capacity, P_t or P_{ca} =		**N/A**	kN	

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Rolled Single T				
Gross area of connected leg, a_1 = B.T =		N/A	cm^2	
Gross area of unconnected leg, $a_2 = A_g - a_1$ =		N/A	cm^2	
Thickness of section at hole, $t_{bolthole,single\ T}$ =		N/A	mm	
Area of section to deduct, $A_{deduct,single\ T} = t_{bolthole,single\ T}.d_{bolthole}.N_{bolthole}$		N/A	cm^2	
Note $A_{deduct,single\ T}$ = 0.0 if welded connection type;				
Net area of connected leg, $a_{n1} = a_1 - A_{deduct,single\ T}$ =		N/A	cm^2	
Net area of unconnected leg, $a_{n2} = a_2$ =		N/A	cm^2	
Effective net area of connected leg, $a_{e1} = K_e.a_{n1}$ (<= a_1) =		N/A	cm^2	
Effective net area of unconnected leg, $a_{e2} = K_e.a_{n2}$ (<= a_2) =		N/A	cm^2	
Effective net area of section, $A_{e,single\ T}$ = [If F_x > 0, $a_{e1}+a_{e2}$ (<		**N/A**	cm^2	
Bolted	P_t or $P_{ca} = p_y(A_{e,single\ T}-0.50a_2)$ =	N/A	kN	*4.6.3.1*
Welded	P_t or $P_{ca} = p_y(A_g-0.30a_2)$ =	N/A	kN	*4.6.3.1*
Axial tension or compression capacity, P_t or P_{ca} =		**N/A**	kN	
Rolled Double Angle				
Gross area of connected leg, a_1 = (B.T or D.t) =		N/A	cm^2	
Gross area of unconnected leg, $a_2 = A_g - a_1$ =		N/A	cm^2	
Thickness of section at hole, $t_{bolthole,double\ angle}$ =		N/A	mm	
Area of section to deduct, $A_{deduct,double\ angle} = t_{bolthole,double\ angle}.d_{bolthole}.$		N/A	cm^2	
Note $A_{deduct,double\ angle}$ = 0.0 if welded connection type;				
Net area of connected leg, $a_{n1} = a_1 - A_{deduct,double\ angle}$ =		N/A	cm^2	
Net area of unconnected leg, $a_{n2} = a_2$ =		N/A	cm^2	
Effective net area of connected leg, $a_{e1} = K_e.a_{n1}$ (<= a_1) =		N/A	cm^2	
Effective net area of unconnected leg, $a_{e2} = K_e.a_{n2}$ (<= a_2) =		N/A	cm^2	
Effective net area of section, $A_{e,double\ angle}$ = [If F_x > 0, 2.(a_{e1}+		**N/A**	cm^2	
Same side - bolted	P_t or $P_{ca} = 2.p_y(A_{e,double\ angle}/2-0.50a_2)$ =	N/A	kN	*4.6.3.2*
Same side - welded	P_t or $P_{ca} = 2.p_y(A_{g,double\ angle}/2-0.30a_2)$ =	N/A	kN	*4.6.3.2*
Both sides - bolted	P_t or $P_{ca} = 2.p_y(A_{e,double\ angle}/2-0.25a_2)$ =	N/A	kN	*4.6.3.2*
Both sides - welded	P_t or $P_{ca} = 2.p_y(A_{g,double\ angle}/2-0.15a_2)$ =	N/A	kN	*4.6.3.2*
Axial tension or compression capacity, P_t or P_{ca} =		**N/A**	kN	
Rolled Double Channel				
Gross area of connected leg, a_1 = D.t =		N/A	cm^2	
Gross area of unconnected leg, $a_2 = A_g - a_1$ =		N/A	cm^2	
Thickness of section at hole, $t_{bolthole,double\ channel}$ =		N/A	mm	
Area of section to deduct, $A_{deduct,double\ channel} = t_{bolthole,double\ channel}.d_{boltl}$		N/A	cm^2	
Note $A_{deduct,double\ channel}$ = 0.0 if welded connection type;				
Net area of connected leg, $a_{n1} = a_1 - A_{deduct,double\ channel}$ =		N/A	cm^2	
Net area of unconnected leg, $a_{n2} = a_2$ =		N/A	cm^2	
Effective net area of connected leg, $a_{e1} = K_e.a_{n1}$ (<= a_1) =		N/A	cm^2	
Effective net area of unconnected leg, $a_{e2} = K_e.a_{n2}$ (<= a_2) =		N/A	cm^2	
Effective net area of section, $A_{e,double\ channel}$ = [If F_x > 0, 2.(a_e		**N/A**	cm^2	
Both sides - bolted	P_t or $P_{ca} = 2.p_y(A_{e,double\ channel}/2-0.25a_2)$ =	N/A	kN	*4.6.3.2*
Both sides - welded	P_t or $P_{ca} = 2.p_y(A_{g,double\ channel}/2-0.15a_2)$ =	N/A	kN	*4.6.3.2*
Axial tension or compression capacity, P_t or P_{ca} =		**N/A**	kN	

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Rolled Double T			
Gross area of connected leg, a_1 = B.T =	N/A	cm^2	
Gross area of unconnected leg, $a_2 = A_g - a_1$ =	N/A	cm^2	
Thickness of section at hole, $t_{bolthole,double\ T}$ =	N/A	mm	
Area of section to deduct, $A_{deduct,double\ T} = t_{bolthole,double\ T}.d_{bolthole}.N_{boltho}$	N/A	cm^2	
Note $A_{deduct,double\ T}$ = 0.0 if welded connection type;			
Net area of connected leg, $a_{n1} = a_1 - A_{deduct,double\ T}$ =	N/A	cm^2	
Net area of unconnected leg, $a_{n2} = a_2$ =	N/A	cm^2	
Effective net area of connected leg, $a_{e1} = K_e.a_{n1}$ (<= a_1) =	N/A	cm^2	
Effective net area of unconnected leg, $a_{e2} = K_e.a_{n2}$ (<= a_2) =	N/A	cm^2	
Effective net area of section, $A_{e,double\ T}$ = [If F_x > 0, 2.(a_{e1}+a_e	**N/A**	cm^2	
Both sides - bolted P_t or $P_{ca} = 2.p_y(A_{e,double\ T}/2-0.25a_2)$ =	N/A	kN	*4.6.3.2*
Both sides - welded P_t or $P_{ca} = 2.p_y(A_{g,double\ T}/2-0.15a_2)$ =	N/A	kN	*4.6.3.2*
Axial tension or compression capacity, P_t or P_{ca} =	**N/A**	kN	
Axial tension or compression capacity, P_t or P_{ca} =	**9699**	kN	
F / (P_t or P_{ca}) utilisation =	**0.143**		**OK**

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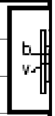
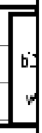
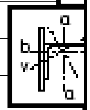
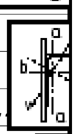
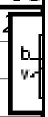
		Member/Location		
Job Title	Member Design - Steel BeamColumn BS5950 v2015.01	Drg. Ref.		
Member Design - Steel BeamColumn		Made by XX	Date 21/11/2021	Chd.

Plastic (or Elastic) Moment Capacity					BS5950
(With Low or High Shear Force; At Least Semi Compact Section)					
	Low shear (V_x <= $0.6P_{vx}$) or high shear (V_x > $0.6P_{vx}$) ?		Low Shear		*4.2.5*
	Low shear (V_y <= $0.6P_{vy}$) or high shear (V_y > $0.6P_{vy}$) ?		Low Shear		*4.2.5*
	Section classification (y-plane where applicable) =		Compact		*4.2.5*
	Section classification (x-plane where applicable) =		Compact		*4.2.5*
	Moment Capacity Case	**M_x**	**M_y**		
1	Low shear and compact =	Valid	Valid		*4.2.5.2*
2	High shear and compact =	Invalid	Invalid		*4.2.5.3*
3	Low shear and semi compact =	Invalid	Invalid		*4.2.5.2*
4	High shear and semi compact =	Invalid	Invalid		*4.2.5.3*
	Moment Capacity (Except Single Angle and Double	**M_x**	**M_y**		
1	Low shear; Compact; $M_c = p_y . s_{relevant}$ =	1540	781	kNm	*4.2.5.2*
2	High shear; Compact; $M_c = p_y . (s_{relevant} - \rho s_{v, relevant})$ =	1308	89	kNm	*4.2.5.3*
3	Low shear; Semi compact; $M_c = p_y . Z_{relevant}$ =	1345	514	kNm	*4.2.5.2*
4	High shear; Semi compact; $M_c = p_y . (Z_{relevant} - \rho s_{v, relevar}$	1190	52	kNm	*4.2.5.3*
	Moment Capacity (Only Single Angle and Double A	**M_x**	**M_y**		
1	Low shear; Compact; $M_c = 0.8 . p_y . Z_{relevant}$ =	N/A	N/A	kNm	*4.2.5.2*
2	High shear; Compact; $M_c = 0.8 . p_y . (Z_{relevant} - \rho s_{v, relevar}$	N/A	N/A	kNm	*4.2.5.3*
3	Low shear; Semi compact; $M_c = 0.8 . p_y . Z_{relevant}$ =	N/A	N/A	kNm	*4.2.5.2*
4	High shear; Semi compact; $M_c = 0.8 . p_y . (Z_{relevant} - \rho s_v$	N/A	N/A	kNm	*4.2.5.3*
	Reduction factor, $\rho = [2(V/P_v) - 1]^2$ =	**1.000**	**1.000**		*4.2.5.3*
	Section	**s_v for M_x**	**s_v for M_y**		
	Rolled I = $t.D^2/4$ and $2.(0.9T.B^2/4)$ =	875	2615	cm^3	*4.2.5.3*
	Rolled RHS = $A_g.D^2/4/(D+B)$ and $2.(0.9T.B^2/4)$ =	N/A	N/A	cm^3	*4.2.5.3*
	Rolled CHS = $0.6.[0.424(D/2).\pi D^2/8-0.424(D/2-t).\pi(D-$	N/A	N/A	cm^3	*4.2.5.3*
	Welded I = $t.(D-2T)^2/4$ and $2.(0.9T.B^2/4)$ =	N/A	N/A	cm^3	*4.2.5.3*
	Welded RHS = $2t.(D-2T)^2/4$ and $2.(0.9T.B^2/4)$ =	N/A	N/A	cm^3	*4.2.5.3*
	Solid Bar = $0.424(D/2).0.9A_g$ =	N/A	N/A	cm^3	*4.2.5.3*
	Plate = $0.9A_g.D/4$ and $0.9A_g.t/4$ =	N/A	N/A	cm^3	*4.2.5.3*
	Rolled Single Angle = $0.9t.D^2/4$ and $(0.9T.B^2/4)$ =	N/A	N/A	cm^3	*4.2.5.3*
	Rolled Single Channel = $t.D^2/4$ and $2.(0.9T.B^2/4)$ =	N/A	N/A	cm^3	*4.2.5.3*
	Rolled Single T = $t.D^2/4$ and $(0.9T.B^2/4)$ =	N/A	N/A	cm^3	*4.2.5.3*
	Rolled Double Angle = $2.(0.9t.D^2/4)$ and $2.(0.9T.B^2/4)$ =	N/A	N/A	cm^3	*4.2.5.3*
	Rolled Double Channel = $2.t.D^2/4$ and $4.(0.9T.B).(s/2+$	N/A	N/A	cm^3	*4.2.5.3*
	Rolled Double T = $2.t.D.(s/2+D/2)$ and $2.(0.9T.B^2/4)$ =	N/A	N/A	cm^3	*4.2.5.3*
	Relevant plastic modulus of shear area, $s_{v, relevant}$ =	**875**	**2615**	cm^3	*4.2.5.3*

				BS5950
Limiting moment capacity, $M_{cx,lim} = 1.2p_y.Z_{x,\ relevant}$ =	1614	kNm		*4.2.5.1*
Limiting moment capacity, $M_{cy,lim} = 1.2p_y.Z_{y,\ relevant}$ =	617	kNm		*4.2.5.1*
Moment capacity, M_{cx} = MIN ($M_{cx,lim}$, M_{cx}) =	**1540**	kNm		*4.2.5*
Moment capacity, M_{cy} = MIN ($M_{cy,lim}$, M_{cy}) =	**617**	kNm		*4.2.5*
M_x / M_{cx} utilisation =	**0.020**			**OK**
M_y / M_{cy} utilisation =	**0.018**			**OK**
MAX (M_x / M_{cx}, M_y / M_{cy}) utilisation =	**0.020**			**OK**


Local Capacity

Note that in compression members, the local capacity may not always be less onerous than the overall buckling capacity because the local capacity includes the effects of section area reduction due to bolt holes and connection connectivity, whereas the overall buckling capacity is based on the gross section area. Clearly in tension members, the local capacity is expected to be more onerous as buckling effects are not applicable;

$$\frac{F_t}{P_t} + \frac{M_x}{M_{cx}} + \frac{M_y}{M_{cy}} \leq 1$$ **Tension**

$$\frac{F_c}{A_g p_y} + \frac{M_x}{M_{cx}} + \frac{M_y}{M_{cy}} \leq 1$$ **Compression**

4.8.
4.8.3.

$$\frac{1387}{9699} + \frac{30}{1540} + \frac{11}{617}$$

0.143 + **0.020** + **0.018** = **0.181** **OK**

R

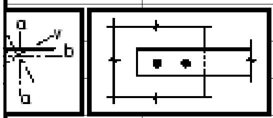
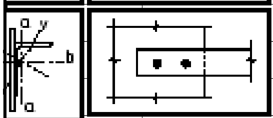
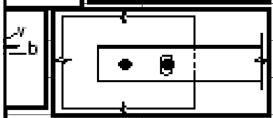
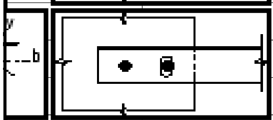
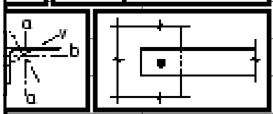
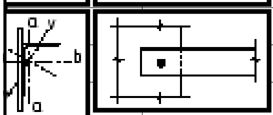
Note $A_g p_y$ above refers to P_{ca};

CONSULTING ENGINEERS	Engineering Calculation Sheet Consulting Engineers	Job No.	Sheet No.	Rev.
		jXXX	17	
		Member/Location		
Job Title	Member Design - Steel BeamColumn BS5950 v2015.01	Drg. Ref.		
Member Design - Steel BeamColumn		Made by XX	Date 21/11/2021	Chd.

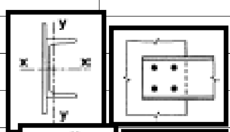
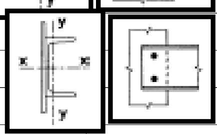
Flexural Buckling (Perry-Robertson) Capacity — <u>BS5950</u>

Slenderness	λ_x		λ_y		λ_v		
Rolled I	L_{ex}/r_x	29.4	L_{ey}/r_y	46.0	N/A	N/A	4.7.2
Rolled RHS	L_{ex}/r_x	N/A	L_{ey}/r_y	N/A	N/A	N/A	4.7.2
Rolled CHS	L_{ex}/r_x	N/A	L_{ey}/r_y	N/A	N/A	N/A	4.7.2
Welded I	L_{ex}/r_x	N/A	L_{ey}/r_y	N/A	N/A	N/A	4.7.2
Welded RHS	L_{ex}/r_x	N/A	L_{ey}/r_y	N/A	N/A	N/A	4.7.2
Solid Bar	L_{ex}/r_x	N/A	L_{ey}/r_y	N/A	N/A	N/A	4.7.2
Plate	L_{ex}/r_x	N/A	L_{ey}/r_y	N/A	N/A	N/A	4.7.2

Rolled Single Angle — *Short side connected - two bolts, standard and kidney clearance*

	As below	N/A	***As below***	N/A	***As below***	N/A	4.7.10.2
	$1.0L_a/r_a$ but $\geq 0.7L_a/r_a + 30$	N/A	$0.85L_b/r_b$ but $\geq 0.7L_b/r_b + 30$	N/A	$0.85L_v/r_v$ but $\geq 0.7L_v/r_v + 15$	N/A	
	$0.85L_b/r_b$ but $\geq 0.7L_b/r_b + 30$	N/A	$1.0L_a/r_a$ but $\geq 0.7L_a/r_a + 30$	N/A	$0.85L_v/r_v$ but $\geq 0.7L_v/r_v + 15$	N/A	
	$1.0L_a/r_a$ but $\geq 0.7L_a/r_a + 30$	N/A	$1.0L_b/r_b$ but $\geq 0.7L_b/r_b + 30$	N/A	$1.0L_v/r_v$ but $\geq 0.7L_v/r_v + 15$	N/A	
	$1.0L_b/r_b$ but $\geq 0.7L_b/r_b + 30$	N/A	$1.0L_a/r_a$ but $\geq 0.7L_a/r_a + 30$	N/A	$1.0L_v/r_v$ but $\geq 0.7L_v/r_v + 15$	N/A	
	$1.0L_a/r_a$ but $\geq 0.7L_a/r_a + 30$	N/A	$1.0L_b/r_b$ but $\geq 0.7L_b/r_b + 30$	N/A	$1.0L_v/r_v$ but $\geq 0.7L_v/r_v + 15$	N/A	
	$1.0L_b/r_b$ but $\geq 0.7L_b/r_b + 30$	N/A	$1.0L_a/r_a$ but $\geq 0.7L_a/r_a + 30$	N/A	$1.0L_v/r_v$ but $\geq 0.7L_v/r_v + 15$	N/A	

lled Single Channel — *Web connected - two or more rows of bolts*

	As below	N/A	***As below***	N/A	N/A	N/A	4.7.1
	$0.85L_x/r_x$	N/A	$1.0L_y/r_y$ but $\geq 0.7L_y/r_y + 30$	N/A	N/A	N/A	
	$1.0L_x/r_x$	N/A	$1.0L_y/r_y$ but $\geq 0.7L_y/r_y + 30$	N/A	N/A	N/A	

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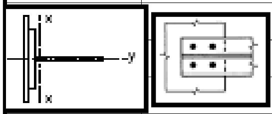
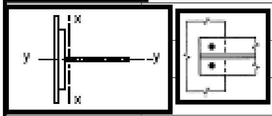
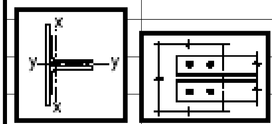
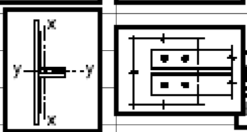
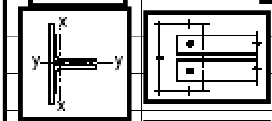
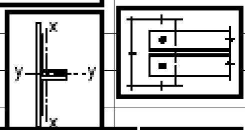
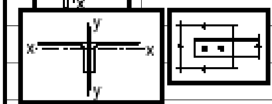
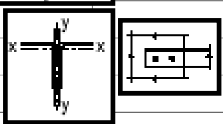
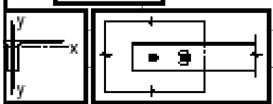
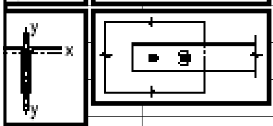
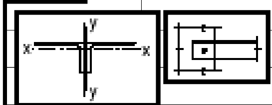
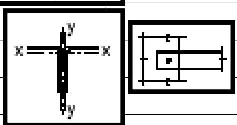
Rolled Single T	*Flange connected - two or more rows of bolts*					Ro
	As below	**N/A**	***As below***	**N/A**	**N/A**	**N/A**	*4.7.10.5*
	$1.0L_x/r_x$ but $\geq 0.7L_x/r_x + 30$	N/A	$0.85L_y/r_y$	N/A	N/A	N/A	
	$1.0L_x/r_x$ but $\geq 0.7L_x/r_x + 30$	N/A	$1.0L_y/r_y$	N/A	N/A	N/A	

Rolled Double Angle *port, short side connected - two or more bolts, standard clearance*

Eleven batten(s) within span L

	As Below	**N/A**	***As Below***	**N/A**	**N/A**	**N/A**	*4.7.10.3*
	$1.0L_x/r_x$ but $\geq 0.7L_x/r_x + 30$	N/A	$[(0.85L_y/r_y)^2 + \lambda_c^2]^{0.5}$ but $\geq 1.4\lambda_c$	N/A	N/A	N/A	
	$[(0.85L_y/r_y)^2 + \lambda_c^2]^{0.5}$ but $\geq 1.4\lambda_c$	N/A	$1.0L_x/r_x$ but $\geq 0.7L_x/r_x + 30$	N/A	N/A	N/A	
	$1.0L_x/r_x$ but $\geq 0.7L_x/r_x + 30$	N/A	$[(L_y/r_y)^2+\lambda_c^2]^{0.5}$ but $\geq 1.4\lambda_c$	N/A	N/A	N/A	
	$[(L_y/r_y)^2+\lambda_c^2]^{0.5}$ but $\geq 1.4\lambda_c$	N/A	$1.0L_x/r_x$ but $\geq 0.7L_x/r_x + 30$	N/A	N/A	N/A	
	$[(L_y/r_y)^2+\lambda_c^2]^{0.5}$ but $\geq 1.4\lambda_c$	N/A	$0.85L_x/r_x$ but $\geq 0.7L_x/r_x + 30$	N/A	N/A	N/A	
	$0.85L_x/r_x$ but $\geq 0.7L_x/r_x + 30$	N/A	$[(L_y/r_y)^2+\lambda_c^2]^{0.5}$ but $\geq 1.4\lambda_c$	N/A	N/A	N/A	
	$[(L_y/r_y)^2+\lambda_c^2]^{0.5}$ but $\geq 1.4\lambda_c$	N/A	$1.0L_x/r_x$ but $\geq 0.7L_x/r_x + 30$	N/A	N/A	N/A	
	$1.0L_x/r_x$ but $\geq 0.7L_x/r_x + 30$	N/A	$[(L_y/r_y)^2+\lambda_c^2]^{0.5}$ but $\geq 1.4\lambda_c$	N/A	N/A	N/A	
	$[(L_y/r_y)^2+\lambda_c^2]^{0.5}$ but $\geq 1.4\lambda_c$	N/A	$1.0L_x/r_x$ but $\geq 0.7L_x/r_x + 30$	N/A	N/A	N/A	
	$1.0L_x/r_x$ but $\geq 0.7L_x/r_x + 30$	N/A	$[(L_y/r_y)^2+\lambda_c^2]^{0.5}$ but $\geq 1.4\lambda_c$	N/A	N/A	N/A	

Note that r_x and r_y above refers to $r_{x, double angle}$ and $r_{y, double angle}$;

Length between adjacent battens, $L_{e, bat}$ = L / (1 + No of battens) =	N/A	m	
Slenderness, $\lambda_c = L_{e, bat} / r_v$ =	N/A		**N/A**

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lled Double Channel *Both sides of support, web connected*

Eleven batten(s) within span L

$L_{ex}/r_{x, \text{double ch}}$	N/A	λ_b	N/A	N/A	N/A	*4.7.9*

$\lambda_b = (\lambda_m^2 + \lambda_c^2)^{0.5}$ If λ_b is less than $1.4\lambda_c$ the design should be based on $\lambda_b = 1.4\lambda_c$.

Length between adjacent battens, $L_{e,\ bat}$ = L / (1 + No of battens) =	N/A	m	
Slenderness, $\lambda_m = L_{ey}/r_{y,\ \text{double channel}}$ =	N/A		
Slenderness, $\lambda_c = L_{e,\ bat} / r_y$ =	N/A		N/A

The slenderness λ_c of a main component (based on its minimum radius of gyration) between end welds or end bolts of adjacent battens λ_c should not exceed 50.

Rolled Double T *Both sides of support, flange connected*

Eleven batten(s) within span L

λ_b	N/A	$L_{ey}/r_{y,\ \text{double}}$	N/A	N/A	N/A	*4.7.9*

$\lambda_b = (\lambda_m^2 + \lambda_c^2)^{0.5}$ If λ_b is less than $1.4\lambda_c$ the design should be based on $\lambda_b = 1.4\lambda_c$.

Length between adjacent battens, $L_{e,\ bat}$ = L / (1 + No of battens) =	N/A	m	
Slenderness, $\lambda_m = L_{ex}/r_{x,\ \text{double T}}$ =	N/A		
Slenderness, $\lambda_c = L_{e,\ bat} / r_y$ =	N/A		N/A

The slenderness λ_c of a main component (based on its minimum radius of gyration) between end welds or end bolts of adjacent battens λ_c should not exceed 50.

Critical slenderness, λ_{max} = MAX (λ_x, λ_y, λ_v) =	**46.0**		
Maximum allowable slenderness, λ_{allow} =	**180.0**		
Slenderness utilisation = λ_{max} / λ_{allow} =	**0.256**		**OK**
Slenderness, λ_x =	**29.4**		
Euler force, $P_{E,x} = \pi^2 EI_{x,\ relevant}/L_{ex}^2$ (valid for $\lambda_x > (\pi^2E/p_y)^{1/2}$) =	85781	kN	
Euler stress, $p_{E,x} = \pi^2E/\lambda_x^2$ (valid for $\lambda_x > (\pi^2E/p_y)^{1/2}$) =	2338	N/mm^2	
Robertson constant, a =	3.5		

The Robertson constant a should be taken as follows:

- for strut curve (a): a = 2.0;
- for strut curve (b): a = 3.5;
- for strut curve (c): a = 5.5;
- for strut curve (d): a = 8.0.

Design strength, p_y =	265	N/mm^2	
Reduced p_y (–20N/mm^2) for plate girders =	265	N/mm^2	
Limiting slenderness, $\lambda_0 = 0.2(\pi^2E/p_y)^{0.5}$ =	17.5		
Perry factor, $\eta_x = a(\lambda_x - \lambda_0)/1000$ (but η_x >= 0) =	0.042		
$\phi_x = [p_y+(\eta_x+1).p_{E,x}] / 2$ =	1351	N/mm^2	
Compressive strength, $p_{cx} = p_{E,x}.p_y / [\phi_x+(\phi_x^2 - p_{E,x}.p_y)^{0.5}]$ =	253	N/mm^2	*4.7.5*
Compression capacity, $P_{cx} = p_{cx}.A_{g,\ relevant}$ =	**9265**	kN	*4.7.4*
F / P_{cx} utilisation =	**0.150**		**OK**

			BS5950
Slenderness, λ_y =	**46.0**		
Euler force, $P_{E,y} = \pi^2 EI_{y,\ relevant}/L_{ey}^2$ (valid for $\lambda_y > (\pi^2 E/p_y)^{1/2}$) =	34830	kN	
Euler stress, $p_{E,y} = \pi^2 E/\lambda_y^2$ (valid for $\lambda_y > (\pi^2 E/p_y)^{1/2}$) =	955	N/mm^2	
Robertson constant, a =	5.5		

The Robertson constant a should be taken as follows:

— for strut curve (a): a = 2.0;
— for strut curve (b): a = 3.5;
— for strut curve (c): a = 5.5;
— for strut curve (d): a = 8.0.

Design strength, p_y =	265	N/mm^2	
Reduced p_y (–20N/mm^2) for plate girders =	265	N/mm^2	
Limiting slenderness, $\lambda_0 = 0.2(\pi^2 E/p_y)^{0.5}$ =	17.5		
Perry factor, $\eta_y = a(\lambda_y - \lambda_0)/1000$ (but $\eta_y >= 0$) =	0.157		
$\phi_y = [p_y+(\eta_y+1).p_{E,y}] / 2$ =	685	N/mm^2	
Compressive strength, $p_{cy} = p_{E,y}.p_y / [\phi_y+(\phi_y^2 - p_{E,y}.p_y)^{0.5}]$ =	220	N/mm^2	*4.7.5*
Compression capacity, $P_{cy} = p_{cy}.A_{g,\ relevant}$ =	**8056**	kN	*4.7.4*
F / P_{cy} utilisation =	**0.172**		**OK**
Slenderness, λ_v =	**N/A**		
Euler force, $P_{E,v} = \pi^2 EI_{v,\ relevant}/L_{ev}^2$ (valid for $\lambda_v > (\pi^2 E/p_y)^{1/2}$) =	N/A	kN	
Euler stress, $p_{E,v} = \pi^2 E/\lambda_v^2$ (valid for $\lambda_v > (\pi^2 E/p_y)^{1/2}$) =	N/A	N/mm^2	
Robertson constant, a =	N/A		

The Robertson constant a should be taken as follows:

— for strut curve (a): a = 2.0;
— for strut curve (b): a = 3.5;
— for strut curve (c): a = 5.5;
— for strut curve (d): a = 8.0.

Design strength, p_y =	N/A	N/mm^2	
Reduced p_y (–20N/mm^2) for plate girders =	N/A	N/mm^2	
Limiting slenderness, $\lambda_0 = 0.2(\pi^2 E/p_y)^{0.5}$ =	N/A		
Perry factor, $\eta_v = a(\lambda_v - \lambda_0)/1000$ (but $\eta_v >= 0$) =	N/A		
$\phi_v = [p_y+(\eta_v+1).p_{E,v}] / 2$ =	N/A	N/mm^2	
Compressive strength, $p_{cv} = p_{E,v}.p_y / [\phi_v+(\phi_v^2 - p_{E,v}.p_y)^{0.5}]$ =	N/A	N/mm^2	*4.7.5*
Compression capacity, $P_{cv} = p_{cv}.A_{g,\ relevant}$ =	**N/A**	kN	*4.7.4*
F / P_{cv} utilisation =	**N/A**		**N/A**
Flexural buckling capacity utilisation =	**0.172**		**OK**

Lateral Torsional Buckling Capacity			BS5950
(With Low or High Shear Force; At Least Semi Compact Section)			
Minor axis slenderness, λ = $L_{E,LTB}/r_{y, relevant}$ =	**46.6**		
Effective unrestrained length, $L_{E,LTB}$ =	4.797	m	4.3.5
β_w = 1 (compact) or β_w = $Z_{x, relevant}/s_{x, relevant}$ (semi compact)	**1.000**		4.3.6.9
Note section classification refers to y-plane where applicable;			
Equivalent slenderness (I, H, single chann... $\lambda_{LT} = uv\lambda\sqrt{\beta_W}$	**32.5**		4.3.6.7
Buckling parameter, u =	0.835		4.3.6.8
Torsional index, x =	10.2		4.3.6.8
λ / x =	4.57		
Slenderness factor, ν = $\nu = \frac{1}{[1+0.05(\lambda/x)^2]^{0.25}}$	0.837		4.3.6.7
Equivalent slenderness (RHS), λ_{LT} = $\lambda_{LT} = 2.25(\phi_b\lambda\beta_W)^{0.5}$	**N/A**		B.2.6
γ_b = $\gamma_b = \left(1-\frac{I_y}{I_x}\right)\left(1-\frac{J}{2.6I_x}\right)$	N/A		B.2.6
ϕ_b = $\phi_b = \left(\frac{S_x^2\gamma_b}{AJ}\right)^{0.5}$	N/A		B.2.6
Note A=A_g in the above equation;			
Equivalent slenderness (CHS, solid bar), λ_{LT} = N/A =	**N/A**		
Equivalent slenderness (plate), λ_{LT} = $\lambda_{LT} = 2.8\left(\frac{\beta_W L_E d}{t^2}\right)^{0.5}$	**N/A**		B.2.7
Note L_E=$L_{E,LTB}$ and d=D in the above eq...			
Equivalent slenderness (single angle, double angle), λ_{LT} = N...	**N/A**		
Equivalent slenderness (single T, double T), λ_{LT} =	**N/A**		B.2.8
Case A: $I_{xx} = I_{yy}$ λ_{LT} is zero	N/A		B.2.8
Case B: $I_{yy} > I_{xx}$ $\lambda_{LT} = 2.8\left(\frac{\beta_W L_E B}{T^2}\right)^{0.5}$	N/A		B.2.8
Case C: $I_{xx} > I_{yy}$ $\lambda_{LT} = uv\lambda\sqrt{\beta_W}$	N/A		B.2.8
Buckling parameter, u =	N/A		B.2.8
Torsional index, x =	N/A		B.2.8
λ / x =	N/A		
$w = \frac{4H}{I_y(D-T/2)^2}$	N/A		B.2.8
Monosymmetric index, ψ =	N/A		B.2.8
$\psi = \left(2y_o - \frac{y_oB^3T/12 + BTy_o^3 + \frac{t}{4}[(c-T)^4-(D-c)^4]}{I_x}\right)\frac{1}{(D-T/2)}$			
Note ψ positive if flange in compression and vice versa;			
Centroid, c = [B.T²/2+...	N/A	mm	B.2.8
$y_o = c - T/2$	N/A	mm	B.2.8
Slenderness factor, ν =	N/A		B.2.8
$\nu = \frac{1}{[(w+0.05(\lambda/x)^2+\psi^2)^{0.5}+\psi]^{0.5}}$			

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Member Design - Steel BeamColumn		Made by **XX**	Date **21/11/2021**	Chd.

				BS5950
Bending strength, p_b =	$p_b = \dfrac{p_E p_y}{\phi_{LT} + (\phi_{LT}^2 - p_E p_y)^{0.5}}$	**265**	N/mm²	*B.2.1*
Slenderness, λ_{LT}		**32.5**		
Euler stress, p_E =	$p_E = (\pi^2 E / \lambda_{LT}^2)$	1912	N/mm²	*B.2.1*
p_y =		265	N/mm²	
ϕ_{LT} =	$\phi_{LT} = \dfrac{p_y + (\eta_{LT} + 1)p_E}{2}$	1089	N/mm²	*B.2.1*
Limiting equivalent slenderness, λ_{L0} =	$0.4(\pi^2 E / p_y)^{0.5}$	35.0		*B.2.1*
Robertson constant, a_{LT} = 7.0 =		7.0		*B.2.1*
Perry factor, η_{LT} =		0.000		*B.2.1*

The Perry factor η_{LT} should be taken as follows:

a) for rolled sections:

$$\eta_{LT} = a_{LT}(\lambda_{LT} - \lambda_{L0})/1\,000 \quad \text{but} \quad \eta_{LT} \geq 0$$

b) for welded sections:

— if $\lambda_{LT} \leq \lambda_{L0}$: $\eta_{LT} = 0$

— if $\lambda_{L0} < \lambda_{LT} < 2\lambda_{L0}$: $\eta_{LT} = 2a_{LT}(\lambda_{LT} - \lambda_{L0})/1\,000$

— if $2\lambda_{L0} \leq \lambda_{LT} \leq 3\lambda_{L0}$: $\eta_{LT} = 2a_{LT}\lambda_{L0}/1\,000$

— if $\lambda_{LT} > 3\lambda_{L0}$: $\eta_{LT} = a_{LT}(\lambda_{LT} - \lambda_{L0})/1\,000$

	Equivalent uniform moment factor for LTB, m_{LT} =	1.000		*4.3.6.6*
	Low shear (V_x <= $0.6P_{vx}$) or high shear (V_x > $0.6P_{vx}$) ?	Low Shear		*4.2.5*
	Section classification (y-plane where applicable) =	Compact		*4.2.5*
	Lateral Torsional Buckling Case	**M_b**		
1	Low shear and compact =	Valid		
2	High shear and compact =	Invalid		
3	Low shear and semi compact =	Invalid		
4	High shear and semi compact =	Invalid		
	Lateral Torsional Buckling (Except Single Angle and Double A	**M_b**		
1	Low shear; Compact; $M_b = p_b \cdot s_{x, relevant}$ =	1540	kNm	*4.3.6.4*
2	High shear; Compact; $M_b = p_b \cdot (s_{x, relevant} - \rho_x s_{vx, relevant})$ =	1308	kNm	*4.3.6.4*
3	Low shear; Semi compact; $M_b = p_b \cdot Z_{x, relevant}$ =	1345	kNm	*4.3.6.4*
4	High shear; Semi compact; $M_b = p_b \cdot (Z_{x, relevant} - \rho_x s_{vx, relevant}/1.5)$ =	1190	kNm	*4.3.6.4*
	Note if p_b not applicable, then equations revert to p_y;			

					BS5950
	Lateral Torsional Buckling (Only Single Angle and	**M_{bx}**	**M_{by}**		
	a. $M_b = 0.8 p_y Z_x$ heel of angle in compression				*4.3.8.3*
	b. $M_b = p_y Z_x \left(\frac{1350\varepsilon - L_E/r_v}{1625\varepsilon}\right)$ but $M_b \le 0.8 p_y Z_x$ heel of angle in tension				*4.3.8.3*
1	a. Low shear; Compact; $M_b = 0.8 . p_y . Z_{relevant}$ =	N/A	N/A	kNm	*4.3.8.3*
	b. Low shear; Compact; $M_b = p_y . Z_{relevant} . (1350\varepsilon\text{-}L_{E,LTB}$	N/A	N/A	kNm	*4.3.8.3*
2	a. High shear; Compact; $M_b = 0.8 . p_y . (Z_{relevant} - \rho s_{v, rel}$	N/A	N/A	kNm	*4.3.8.3*
	b. High shear; Compact; $M_b = p_y . (Z_{relevant} - \rho s_{v, relevant})$	N/A	N/A	kNm	*4.3.8.3*
3	a. Low shear; Semi compact; $M_b = 0.8 . p_y . Z_{relevant}$ =	N/A	N/A	kNm	*4.3.8.3*
	b. Low shear; Semi compact; $M_b = p_y . Z_{relevant} . (1350\varepsilon$	N/A	N/A	kNm	*4.3.8.3*
4	a. High shear; Semi compact; $M_b = 0.8 . p_y . (Z_{relevant}$ -	N/A	N/A	kNm	*4.3.8.3*
	b. High shear; Semi compact; $M_b = p_y . (Z_{relevant} - \rho s_{v, rel}$	N/A	N/A	kNm	*4.3.8.3*
	Note although the equations are valid only for equal angles, it is used for unequal angles too;				
	Reduction factor, $\rho_x = [2(V_x/P_{vx}) - 1]^2$ =		**1.000**		*4.2.5.3*
	Reduction factor, $\rho_y = [2(V_y/P_{vy}) - 1]^2$ =		**N/A**		*4.2.5.3*
	Relevant plastic modulus of shear area, $s_{vx, relevant}$ =		**875**	cm^3	*4.2.5.3*
	Relevant plastic modulus of shear area, $s_{vy, relevant}$ =		**2615**	cm^3	*4.2.5.3*
	Moment capacity, M_{cx} =		**1540**	kNm	*4.2.5*
	Moment capacity, M_{cy} =		**N/A**	kNm	*4.2.5*
	Lateral torsional buckling capacity, M_b/m_{LT} (or M_{bx}/m_{LT}) =		**1540**	kNm	*4.3.6.2*
	Lateral torsional buckling capacity, M_{by}/m_{LT} =		**N/A**	kNm	*4.3.6.2*
	Lateral torsional buckling capacity, MIN (M_b/m_{LT} (or M_{bx}/m_{L}		**1540**	kNm	*4.3.6.2*
	Lateral torsional buckling capacity, MIN (M_{by}/m_{LT}, M_{cy}) =		**N/A**	kNm	*4.3.6.2*
	M_x / MIN (M_b/m_{LT} (or M_{bx}/m_{LT}), M_{cx}) utilisation =		**0.020**		**OK**
	M_y / MIN (M_{by}/m_{LT}, M_{cy}) utilisation =		**N/A**		**N/A**
	MAX (M_x / MIN (M_b/m_{LT} (or M_{bx}/m_{LT}), M_{cx}), M_y / MIN (M_{by}/m		**0.020**		**OK**

Overall Buckling (Simplified Approach) Capacity — BS5950

Equivalent uniform moment factor for flexural buckling, m_x =	1.000	*4.8.3.3.4*
Equivalent uniform moment factor for flexural buckling, m_y =	1.000	*4.8.3.3.4*

Overall Buckling (Except Single Angle and Double Angle)

$\frac{F_c}{P_{cy}} + \frac{m_{LT}M_{LT}}{M_b} + \frac{m_y M_y}{p_y Z_y} \le 1$	0.172 + 0.020 + 0.022 =	0.214	*4.8.3.3.4*
$\frac{F_c}{P_c} + \frac{m_x M_x}{p_y Z_x} + \frac{m_y M_y}{p_y Z_y} \le 1$	0.172 + 0.023 + 0.022 =	0.217	*4.8.3.3.4*

Note Z_x and Z_y refers to $Z_{x, relevant}$ and $Z_{y, relevant}$, respectively;

Overall Buckling (Only Single Angle and Double Angle)

$\frac{F_c}{P_c} + \frac{m_{LTx}M_x}{M_{bx}} + \frac{m_{LTy}M_y}{M_{by}} \le 1$	N/A + N/A + N/A =	N/A	*I.4.3*

Note although the equations are valid only for equal angles, it is used for unequal angles too;

Note m_{LTx} and m_{LTy} both taken as m_{LT};

Overall buckling utilisation = 0.217 **OK**

Overall Buckling (Exact Approach) Capacity

Equivalent uniform moment factor for flexural buckling, m_{yx} =	1.000	*4.8.3.3.4*

Overall Buckling (Only I and H)

— for major axis buckling: $\frac{F_c}{P_{cx}} + \frac{m_x M_x}{M_{cx}}\left(1 + 0.5\frac{F_c}{P_{cx}}\right) + 0.5\frac{m_{yx}M_y}{M_{cy}} \le 1$	0.180	*4.8.3.3.2*
— for lateral-torsional buckling: $\frac{F_c}{P_{cy}} + \frac{m_{LT}M_{LT}}{M_b} + \frac{m_y M_y}{M_{cy}}\left(1 + \frac{F_c}{P_{cy}}\right) \le 1$	0.214	*4.8.3.3.2*
— for interactive buckling: $\frac{m_x M_x(1 + 0.5(F_c/P_{cx}))}{M_{cx}(1 - F_c/P_{cx})} + \frac{m_y M_y(1 + F_c/P_{cy})}{M_{cy}(1 - F_c/P_{cy})} \le 1$	0.051	*4.8.3.3.2*

Overall Buckling (Only RHS and CHS)

— for major axis buckling: $\frac{F_c}{P_{cx}} + \frac{m_x M_x}{M_{cx}}\left(1 + 0.5\frac{F_c}{P_{cx}}\right) + 0.5\frac{m_{yx}M_y}{M_{cy}} \le 1$	N/A	*4.8.3.3.3*
— for minor axis buckling, if a lateral-torsic $\frac{F_c}{P_{cy}} + \frac{m_{LT}M_{LT}}{M_b} + \frac{m_y M_y}{M_{cy}}\left(1 + 0.5\frac{F_c}{P_{cy}}\right) \le 1$	N/A	*4.8.3.3.3*
— for interactive buckling: $\frac{m_x M_x(1 + 0.5(F_c/P_{cx}))}{M_{cx}(1 - F_c/P_{cx})} + \frac{m_y M_y(1 + 0.5(F_c/P_{cy}))}{M_{cy}(1 - F_c/P_{cy})} \le$	N/A	*4.8.3.3.3*

Overall buckling utilisation = 0.214 **OK**

Bending Deflection in Y-plane (Under Unfactored Live Load Only)				BS5950
Support for deflections =		Simply supported		
Uniformly distributed unfactored live load, ω_{LL} =		0.0	kN/m	
Deflection, δ_{LL} =		**0.0**	mm	
Simply supported	$\delta_{LL} = 5\,\omega_{LL} L^4/384EI_{x,\ relevant}$ =	0.0	mm	
Cantilever	$\delta_{LL} = \omega_{LL} L^4/8EI_{x,\ relevant}$ =	0.0	mm	
Continuous	$\delta_{LL} = \omega_{LL} L^4/384EI_{x,\ relevant}$ =	0.0	mm	
Continuous end span	$\delta_{LL} = \omega_{LL} L^4/185EI_{x,\ relevant}$ =	0.0	mm	
Max deflection limit =		**8.9**	mm	*2.5.2*
Simply supported	*span / 360 =*	8.9	mm	
Cantilever	*span / 180 =*	17.8	mm	
Continuous	*span / 360 =*	8.9	mm	
Continuous end span	*span / 360 =*	8.9	mm	
Deflection utilisation = δ_{LL} / max deflection limit =		**0.000**		**OK**
Bending Deflection in Y-plane (Under SLS Load)				
Support for deflections =		Simply supported		
Uniformly distributed SLS load, ω_{SLS} =		0.0	kN/m	
Deflection, δ_{SLS} =		**0.0**	mm	
Simply supported	$\delta_{SLS} = 5\,\omega_{SLS} L^4/384EI_{x,\ relevant}$ =	0.0	mm	
Cantilever	$\delta_{SLS} = \omega_{SLS} L^4/8EI_{x,\ relevant}$ =	0.0	mm	
Continuous	$\delta_{SLS} = \omega_{SLS} L^4/384EI_{x,\ relevant}$ =	0.0	mm	
Continuous end span	$\delta_{SLS} = \omega_{SLS} L^4/185EI_{x,\ relevant}$ =	0.0	mm	
Percentage of dead and superimposed dead load deflection precam		0.0	%	
Dead and superimposed dead load deflection precamber, %		**0.0**	mm	
Deflection with precamber incorporated, δ_{SLS} - %pcam . (δ_{SLS}		**0.0**	mm	
Max deflection limit =		**12.8**	mm	*2.5.2*
Simply supported	*span / 250 =*	12.8	mm	
Cantilever	*span / 125 =*	25.6	mm	
Continuous	*span / 250 =*	12.8	mm	
Continuous end span	*span / 250 =*	12.8	mm	
Deflection utilisation = δ_{SLS} / max deflection limit =		**0.000**		**OK**

	Cell References								
	Steel Grade				1	1			
	S275 (43)	265	265						
	S355 (50)	345	345						
	S460 (55)	440	440						
	Section					87			
	Support for Deflections					1			
	Simply supported								
	Cantilever								
	Continuous								
	Continuous end span								
	Section Type and Process					1			
1	Rolled I								
2	Rolled RHS								
3	Rolled CHS								
4	Welded I								
5	Welded RHS								
6	Solid Bar								
7	Plate								
8	Rolled Single Angle								
9	Rolled Single Channel								
10	Rolled Single T								
11	Rolled Double Angle								
12	Rolled Double Channel								
13	Rolled Double T								
	Hot Finished or Cold Formed Rolled RHS and Rolle					1			
	Hot finished								
	Cold formed								
	Rolled Double Angle Orientation					3			
	One side of support, short side connected								
	One side of support, long side connected								
	Both sides of support, short side connected								
	Both sides of support, long side connected								
	Rolled Double Channel Orientation					1			
	Both sides of support, web connected								
	Rolled Double T Orientation					1			
	Both sides of support, flange connected								
	Bolted or Welded Connection Type					2			
	Bolted								
	Welded								
	Rolled I and Welded I Axial Capacity Connection C					4			
	Web connected								
	Both flanges connected								
	Web and single flange connected								
	Web and both flanges connected								

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		Member/Location		
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Member Design - Steel BeamColumn		Made by XX	Date 21/11/2021	Chd.

Web Bearing and Buckling				BS5950
Applicability of check for particular section ?		**Applicable**		
Local compressive force, $F_x = V_x$ =		**0**	kN	
Note F_x is the reaction or shear force at the supports of simply supported beams, externally				
applied loads or the reaction (not shear force) at internal supports of continuous beams;				
Unstiffened web bearing and buckling capacity utilisation =		**0.000**		**OK**
Stiffened web bearing and buckling capacity utilisation =		**0.000**		**OK**
Web stiffener steel grade (usually grade 43) =		S275 (43)		
Web stiffener design strength, p_{ys} =		265	N/mm^2	

Section Type	**N_1**	**N_2**		**d**	
Rolled I	1	2	$d = D-2T-2r_i$ =	290.2	mm
Rolled RHS	N/A	N/A	d = D-3or5T =	N/A	mm
Rolled CHS	N/A	N/A	N/A =	N/A	mm
Welded I	N/A	N/A	d = D-2T =	N/A	mm
Welded RHS	N/A	N/A	d = D-2T =	N/A	mm
Solid Bar	N/A	N/A	N/A =	N/A	mm
Plate	N/A	N/A	N/A =	N/A	mm
Rolled Single Angle	N/A	N/A	d = D =	N/A	mm
Rolled Single Channel	N/A	N/A	$d = D-2T-2r_i$ =	N/A	mm
Rolled Single T	N/A	N/A	d = D =	N/A	mm
Rolled Double Angle	N/A	N/A	d = D =	N/A	mm
Rolled Double Channe	N/A	N/A	$d = D-2T-2r_i$ =	N/A	mm
Rolled Double T	N/A	N/A	d = D =	N/A	mm

Number of webs, N_1 =	**1**		
Number of sides for each web, N_2 =	**2**		
Web depth, d =	**290.2**	mm	
Stiff bearing length (along length of web), b_1 =	300	mm	
Continuous over bearing or end bearing ?	End bearing		
Distance, b_e =	0	mm	
Distance, $a_e = b_e + b_1/2$ =	150	mm	
Note b_e is the distance from the end of stiff bearing to the nearer member end;			
Note a_e is the distance from the centre of load or reaction to the nearer member end;			
Number of web stiffeners, N_s =	3		
Note web stiffeners at each cross section is considered as one web stiffener, even if			
separated by multiple webs and/or multiple sides of webs;			
Thickness of web stiffener, t_s =	20.0	mm	
Total length of web stiffener per cross section, b_s =	450	mm	
Note for double angles of case 2 and case 3 and for double T sections, b_s is doubled;			
Outstand length of web stiffener, $b_{s,o} = b_s / (N_1.N_2)$ =	225	mm	
Outstand of web stiffener length limit utilisation, $b_{s,o}$ (<= 13	**0.850**		**OK**
Note that the effectiveness of the outstand of the stiffener is limited to 13 ε t_s;			*4.5.1.2*
Note for channel sections, heel radius reduction factors K_b and K_w have been ignored;			

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Member Design - Steel BeamColumn		Made by XX	Date 21/11/2021	Chd.

BS5950

A. Bearing Capacity of Unstiffened Web

Bearing capacity, P_{bw} = $P_{bw} = (b_1 + nk)tp_{yw}$ **2416** kN

Note t in the above P_{bw} equation is multiplied by N_1;

Stiff bearing length (along length of web), b_1 =	300	mm
Factor, n =	2.0	
Continuous over bearing, n = 5.0 =	N/A	
End bearing, n = 2.0 + 0.6b_e/k (<=5.0) =	2.0	
Factor, k =	**51.7**	mm
Root radius, r_i =	15.2	mm

Section Type	**Factor, k**		
Rolled I	T + r_i =	51.7	mm
Rolled RHS	t =	N/A	mm
Rolled CHS	N/A =	N/A	mm
Welded I	T =	N/A	mm
Welded RHS	t =	N/A	mm
Solid Bar	N/A =	N/A	mm
Plate	N/A =	N/A	mm
Rolled Single Angle	T + r_i =	N/A	mm
Rolled Single Channel	T + r_i =	N/A	mm
Rolled Single T	T + r_i =	N/A	mm
Rolled Double Angle	T + r_i =	N/A	mm
Rolled Double Channel	T + r_i =	N/A	mm
Rolled Double T	T + r_i =	N/A	mm

Unstiffened web bearing capacity utilisation = F_x / P_{bw} = **0.000** **OK**

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The stiff bearing length b_1 should be taken as the length of support that cannot deform appreciabl[y] bending. To determine b_1 the dispersion of load through a steel bearing should be taken as indicat[e] Figure 13. Dispersion at 45° through packs may be included provided that they are firmly fixed in [p]

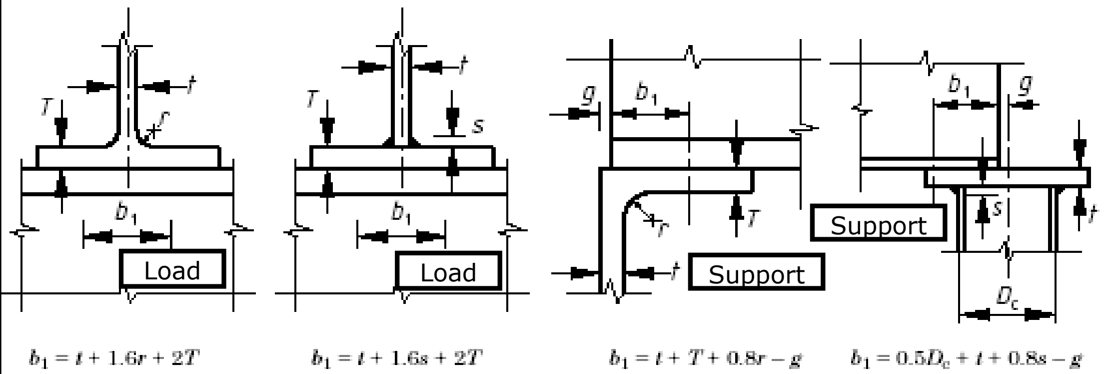
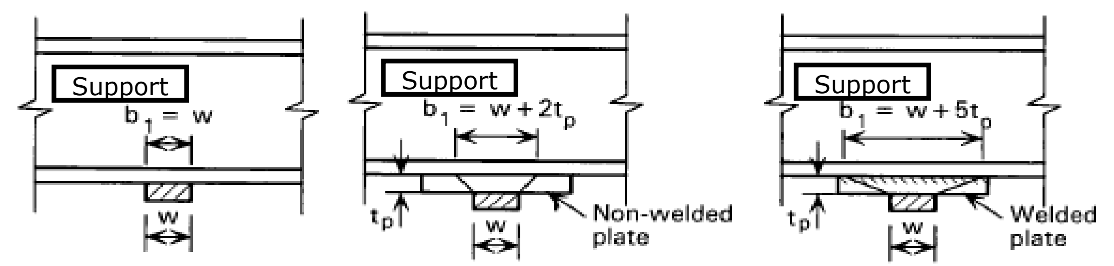


Figure 13 — Stiff bearing length



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y in
ed in
place.

B. Bearing Capacity of Stiffened Web

Note (each) web stiffener refers to stiffeners both sides (if applicable) of each web, but not of all webs (if applicable);

Required (each) web stiffener bearing capacity, MAX(0, F_x -	**0**	kN	
(Each) web stiffener bearing capacity, P_s = $P_s = A_{s,net} p_y$	**2385**	kN	
Area of (each) web stiffener, $A_{s,net}$ = t_s.b_s/N_1 =	90	cm^2	
Stiffened web bearing capacity utilisation = [MAX(0, F_x - P_{bw}	**0.000**		**OK**

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C. Buckling Capacity of Unstiffened Web

Buckling capacity, P_x = **2473** kN

Unstiffened web buckling capacity utilisation = F_x / $P_{x/xr}$ = **0.000** **OK**

C.(i) Section(s) Rolled I, Welded I, Rolled Single Angle, Rolled Single Channel Rolled Single T, Rolled Double Angle, Rolled Double Channel, Rolled Double T

Applicability ? **Applicable**

Buckling capacity, $P_{x/xr}$ = Case B: Reduced buckling capacity **2473** kN

Case A: Unreduced buckling capacity, P_x = **3533** kN

If the flange through which the load or reaction is applied is effectively restrained against both:
a) rotation relative to the web;
b) lateral movement relative to the other flange;

Continuous over bearing or end bearing (case a_e >= 0.7d)

$$P_x = \frac{25\varepsilon t}{\sqrt{(b_1 + nk)d}} P_{bw}$$ N/A kN

End bearing (case a_e < 0.7d)

$$P_x = \frac{a_e + 0.7d}{1.4d} \frac{25\varepsilon t}{\sqrt{(b_1 + nk)d}} P_{bw}$$ 3533 kN

Note t in the above P_x equation need not multiplied by N_1 since already incorporated within P_{bw};

Case B: Reduced buckling capacity, P_{xr} = $$P_{xr} = \frac{0.7d}{L_E} P_x$$ **2473** kN

Effective length, $L_{E,w}$ = fu Non-sway: 1.00d 290.2 mm

Table 22 — Nominal effective length L_E for a compression member[a]

a) non-sway mode			
Restraint (in the plane under consideration) by other parts of the structure			L_E
Effectively held in position at both ends	Effectively restrained in direction at both ends		0.7L
	Partially restrained in direction at both ends		0.85L
	Restrained in direction at one end		0.85L
	Not restrained in direction at either end		1.0L
b) sway mode			
One end	Other end		L_E
Effectively held in position and restrained in direction	Not held in position	Effectively restrained in direction	1.2L
		Partially restrained in direction	1.5L
		Not restrained in direction	2.0L

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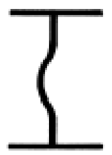
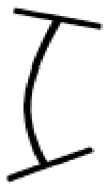
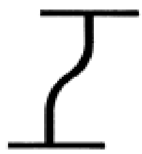
Table K.1 — Effective lengths and slenderness ratios of an unstiffened web acting as a column

Diagrammatic representation of deformation	Restraint conditions	Effective length, l	Slenderness ratio, l/r
	Web ends restrained against both rotation and relative lateral movement	$0.7D$	$2.4D/t_w$
	Web ends restrained against relative lateral movement but not against rotation	$1.0D$ (but see Note 2)	$3.5D/t_w$
	Web ends restrained against rotation but not against relative lateral movement	$1.2D$	$4.2D/t_w$
	One web end not restrained against rotation nor against relative lateral movement and the other web end restrained against both rotation and relative lateral movement	$2.0D$ (but see Note 2)	$7.0D/t_W$

NOTE 1

l the effective length (in mm);

D is the overall depth of the section (in mm);

r is the radius of gyration (in mm);

t_w is the web thickness (in mm).

NOTE 2 Where the ends are not restrained against rotation, l/r should be based on the distance between the effective centres of rotation, which may necessitate taking effective lengths greater than $1.0D$ or $2.0D$.

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C.(ii) Section(s) Rolled RHS, Welded RHS

Applicability ?	**N/A**	
Buckling capacity, $P_{x/xr}$ = Case B: Reduced buckling capacity	**N/A**	kN
Case A: Unreduced buckling capacity, $P_x = (b_1 + n_1)\ 2\ t\ p_c$	**N/A**	kN
Stiff bearing length (along length of web), b_1 =	N/A	mm
Length, $n_1 = b_1+D/2+MIN((a_e-b_1/2),D/2)$ =	N/A	mm

Note n_1 is the length obtained by 45 ° dispersion at half depth (along length of web);

Note for continuous over bearing the expression $MIN((a_e-b_1/2),D/2)$ becomes D/2;

Slenderness, λ_w = $\lambda = 1.5\left[\frac{D-2t}{t}\right]\sqrt{3}$	N/A	
Euler stress, $p_{E,w} = \pi^2E/\lambda_w^2$ (valid for $\lambda_w > (\pi^2E/p_y)^{1/2}$) =	N/A	N/mm^2
Robertson constant, a = (curve (c)) = 5.5 =	N/A	
Design strength, p_y =	N/A	N/mm^2
Limiting slenderness, $\lambda_0 = 0.2(\pi^2E/p_y)^{0.5}$ =	N/A	
Perry factor, $\eta_w = a(\lambda_w-\lambda_0)/1000$ (but η_w >=	N/A	
$\phi_w = [p_y+(\eta_w+1).p_{E,w}] / 2$ =	N/A	N/mm^2
Compressive strength, $p_{cw} = p_{E,w}.p_y / [\phi_w+(\phi_w^2-p_{E,w}.p_y)^0$	N/A	N/mm^2
Case B: Reduced buckling capacity, P_{xr} = 4P + 4P/D.b_1 =	**N/A**	kN

Unless loads or reactions are applied through welded flange plates, the additional effects of moments in the web due to eccentric loading must be taken into account, which will result in lower buckling values.

$e = 0.026B + 0.978t + 0.002D$	N/A	mm
M/P = $M = Pe\left[\frac{B-t-e}{B-t}\right]$	N/A	mm
$a = D/B$	N/A	mm
M_y/P = $M_y = \frac{0.5M\ (3+a)}{a^2+4a+3}$	N/A	mm
$P = \frac{Dt^2p_yp_c}{2tp_y+12\left[M_y/P\right]p_c}$	N/A	kN

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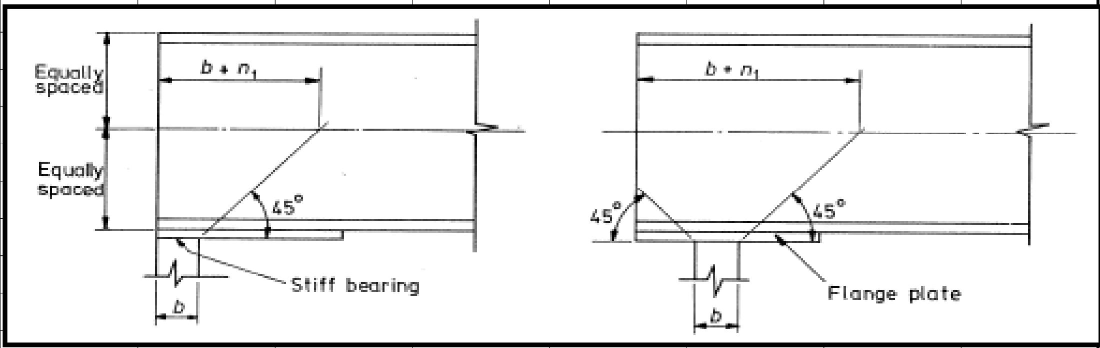

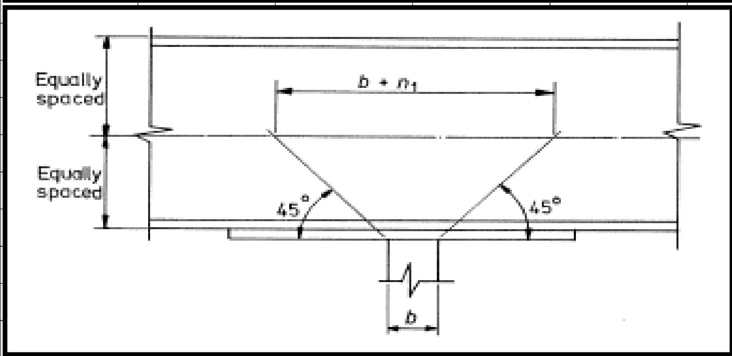

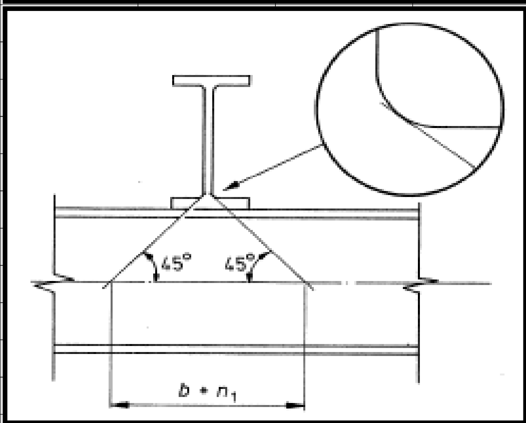

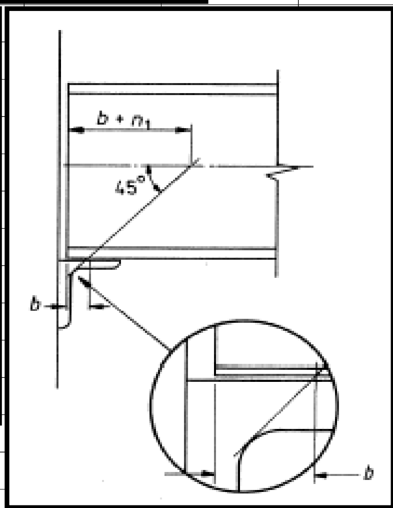

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D. Buckling Capacity of Stiffened Web

Note (each) web stiffener refers to stiffeners both sides (if applicable) of each web, but not of all webs (if applicable);

Required (each) web stiffener buckling capacity, IF($F_x/P_{x/xr}$	**0**	kN	
(Each) web stiffener buckling capacity, P_x = $P_x = A_s\,p_c$	**4415**	kN	
Interior or end web stiffener, n_{IE} = End	1		
Note n_{IE} determines the extent of web which is to contribute to the buckling capacity by affecting terms within A_s and I_s;			
Area of (each) web stiffener, $A_s = t_s.b_s/N_1 + t.n_{IE}.15t$ =	167	cm^2	
Slenderness, $\lambda_s = L_{E,s}/r_s$ =	2.1		
Web depth, d =	290.2	mm	
Effective length, $L_{E,s}$ = fu Non-sway: 0.70d	203.1	mm	
Second moment of area of (each) web stiffe	15220	cm^4	
Note $I_s = 1/12.t_s.(b_s/N_1)^3 + 1/12.n_{IE}.15t.t^3$;			
Note calculation of I_s is simplified, but conservative;			
Radius of gyration, $r_s = \sqrt{(I_s/A_s)}$ =	9.6	cm	
Euler force, $P_{E,s} = \pi^2 EI_s/L_{E,s}^2$ (valid for $\lambda_s > (\pi^2E/p_y)^{1/2}$) =	7462434	kN	
Euler stress, $p_{E,s} = \pi^2E/\lambda_s^2$ (valid for $\lambda_s > (\pi^2E/p_y)^{1/2}$) =	447888	N/mm^2	
Robertson constant, a = (curve (c)) = 5.5 =	5.5		
Design strength, p_{ys} =	265	N/mm^2	
Limiting slenderness, $\lambda_0 = 0.2(\pi^2E/p_{ys})^{0.5}$ =	17.5		
Perry factor, $\eta_s = a(\lambda_s-\lambda_0)/1000$ (but η_s >= (	0.000		
$\phi_s = [p_{ys}+(\eta_s+1).p_{E,s}] / 2$ =	224076	N/mm^2	
Compressive strength, $p_{cs} = p_{E,s}.p_{ys} / [\phi_s+(\phi_s^2-p_{E,s}.p_{ys})^{0.}$	265	N/mm^2	
Stiffened web buckling capacity utilisation = IF($F_x/P_{x/xr}$>1,	**0.000**		**OK**

Scheme Design | *BS5950*

Element	Typical Span/depth	Typical Span (m)
Floor Beams (UB's) (including floor slab)	15-18	up to 12m
Plate girder	10-12	
Slimfloor (steel only)	25-28	6-9m
Castellated UB's*	14-17	12-20m
Lattice girders (RSA's)+	12-15	up to 35m
Lattice girders (Tubular)	15-18	up to 100m
Roof trusses (pitch>20°)	14-15	up to 17m
Space Frames	20-24	up to 60m

* Avoid if high point loads; increase Ireq by 1.3

\+ Precamber by L/250

Beam type	Span range (m)	Notes
(0) Angles	3–6	Used for roof purlins, sheeting rails, etc., where only light loads have to be carried
(1) Cold-formed sections	4–8	Used for roof purlins, sheeting rails, etc., where only light loads have to be carried
(2) Rolled sections: UBs, UCs, RSJs, RSCs	1–30	Most frequently used type of section; proportions selected to eliminate several possible types of failure
(3) Open web joists	4–40	Prefabricated using angles or tubes as chords and round bar for web diagonals, used in place of rolled sections
(4) Castellated beams	6–60	Used for long spans and/or light loads; depth of UB increased by 50%; web openings may be used for services, etc.
(5) Compound sections e.g. UB + RSC	5–15	Used when a single rolled section would not provide sufficient capacity; often arranged to provide enhanced horizontal bending strength as well
(6) Plate girders	10–100	Made by welding together 3 plates sometimes automatically; web depths up to 3–4 m sometimes need stiffening
(7) Trusses	10–100	Heavier version of (3); may be made from tubes, angles or if spanning large distances, rolled sections
(8) Box girders	15–200	Fabricated from plate, usually stiffened; used for OHT cranes and bridges due to good torsional and transverse stiffness properties

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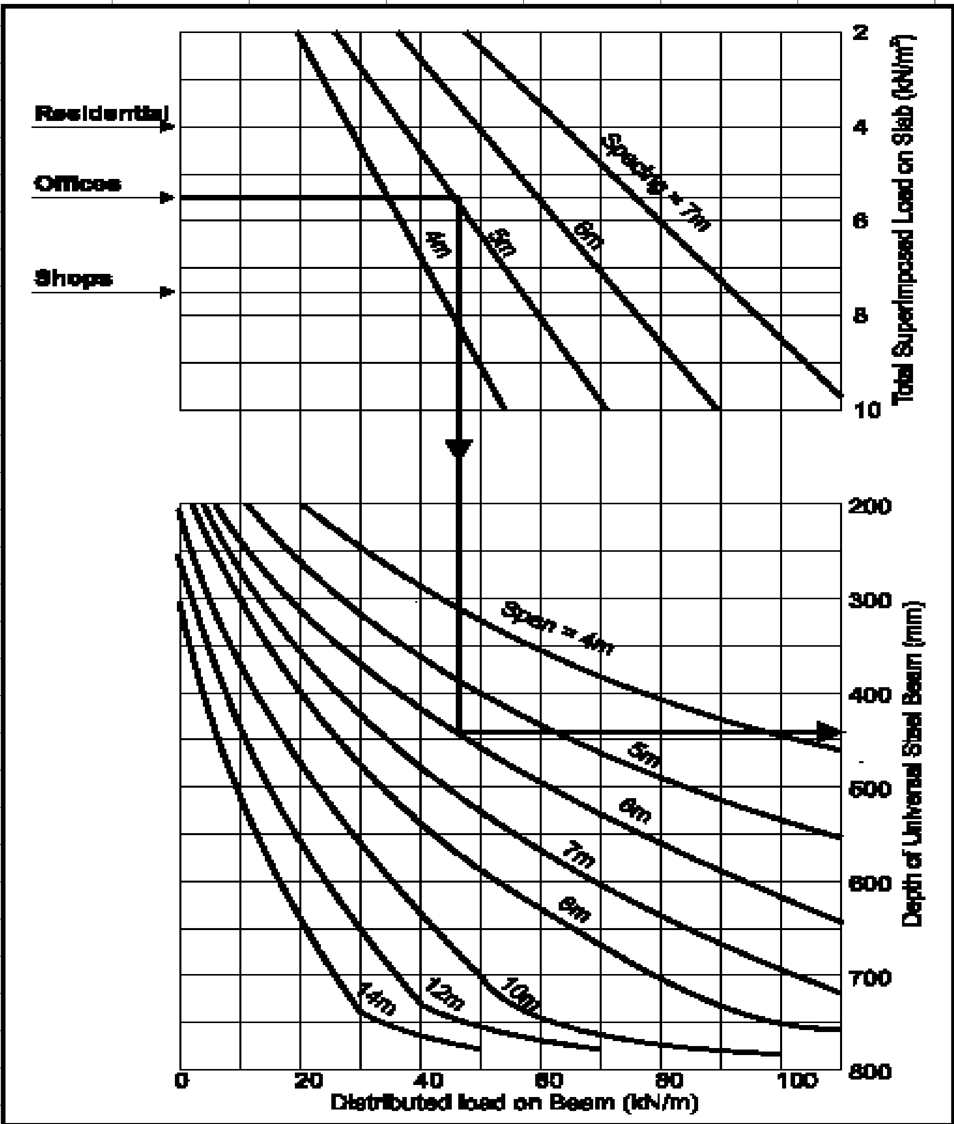

Columns

Preliminary design based on a concentric axial load (see section 4.4.4).

For <u>top</u> storey:
Prelim. design axial load = total axial load + 4 × difference in Y-Y axis load + 2 × difference in X-X axis load

For <u>intermediate</u> storey:
Prelim. design axial load = total axial load + 2 × difference in Y-Y axis load + 1 × difference in X-X axis load

Typical maximum column sizes for braced frames:

- 203 UC for buildings up to 3 storeys high.
- 254 UC for buildings up to 5 storeys high.
- 305 UC for buildings up to 8 storeys high.
- 356 UC for buildings from 8 to 12 storeys high.

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Struts and ties

Slenderness limits:
(No longer in BS5950 (2000) but still of interest)

- members resisting load other than wind: $\lambda \leq 180$
- members resisting self weight and wind only: $\lambda \leq 250$
- members normally acting as a tie but subject to load reversal due to wind: $\lambda \leq 350$

Minimum CHS sections which satisfy slenderness limits

Slenderness Limit	Effective Length (m)				
	4	6	8	10	12
180	76.1 x 3.2	114.3 x 3.6	139.7 x 5.0	168.3 x 5.0	193.7 x 5.0
250	60.6 x 3.2	76.1 x 3.2	114.3 x 3.6	139.7 x 5.0	139.7 x 5.0
350	42.2 x 4.6	60.3 x 3.2	76.1 x 3.2	88.9 x 3.2	114.3 x 3.6

BEAM DESIGN

Ultimate strength in bending

Compression flange restrained

Plastic & Compact

$M_{cx} = p_y S_x \leq 1.5\, p_y Z_x$
(simply supported + cantilever)

$M_{cx} = p_y S_x \leq 1.2\, p_y Z_x$
(continuous)

Semi-compact

$M_{cx} = p_y Z$ *
($S_x > S_{eff} > Z_x$)
*Note: Code allows S_{eff} to be used instead of Z for I or H sections, but this must be calculated.

Requirement :

$M_{cx} \geq M_{max}$

Compression flange unrestrained:

$M_b = p_b S_x$ (plastic & compact)
$M_b = p_b Z$ (semi-compact)

Note : M_b obtained directly from graph (P.5/23)

Requirement :

$M_b \geq m M_{max}$ (for beam not loaded between restrained points)

where: $$m_{LT} = 0.2 + \frac{15M_2 + 0.5M_3 + 15M_4}{M_{max}}$$

but: $m_{LT} \geq 0.44$

The moments M_2 and M_4 are the values at the quarter points and the moment M_3 is the value at mid-length.

BENDING

Universal Beams	GRADE 43						GRADE 50						
D×b×Mass (mm×mm ×Kg/m)	M_{cx} kNm	L_1 m (1.0)	L_2 m (0.75)	L_3 m (0.5)	L_4 m (0.35)	P_v kN	M_{cx} kNm	L_1 m (1.0)	L_2 m (0.75)	L_3 m (0.5)	L_4 m (0.35)	P_v kN	Intermediate masses (kg/m)
914×419×388	4680	3.9	7.7	12.5	-	3150	6020	3.4	6.8	10.8	15.0	4100	
914×419×343	4100	3.8	7.3	12.0	-	2810	5270	3.4	6.7	10.5	14.4	3660	
914×305×289	3340	2.7	5.1	8.2	11.5	2890	4280	2.4	4.5	7.5	10.1	3760	253, 224
914×305×201	2220	2.5	4.7	7.2	9.7	2180	2840	2.2	4.3	6.4	8.5	2840	
838×292×226	2430	2.5	4.8	7.7	10.7	2180	3110	2.3	4.3	6.8	9.2	2840	194
838×292×176	1800	2.4	4.6	7.0	9.4	1860	2320	2.1	4.2	5.3	8.2	2420	
762×267×197	1900	2.4	4.6	7.1	9.9	1910	2440	2.1	4.0	6.2	8.6	2490	173
762×267×147	1370	2.2	4.3	6.4	8.6	1550	1760	2.0	3.7	5.7	7.8	2010	
686×254×170	1490	2.3	4.3	6.9	9.7	1600	1910	2.0	4.1	6.1	8.4	2080	152, 140
686×254×125	1060	2.1	4.0	6.3	8.3	1260	1360	1.9	3.7	5.6	7.3	1640	
610×305×238	1980	3.0	6.0	10.2	15.0	1870	2540	2.6	5.3	9.0	13.0	2440	179
610×305×149	1460	2.8	5.6	9.0	13.0	1150	1550	2.5	4.9	7.5	10.3	1500	
610×229×140	1100	2.1	3.9	6.3	9.0	1290	1410	1.8	3.5	5.6	7.7	1670	125, 113
610×229×101	794	1.9	3.6	5.5	7.5	1050	1020	1.7	3.3	5.0	6.6	1360	
533×210×122	849	1.9	3.7	6.1	8.1	1110	1090	1.7	3.3	5.3	7.3	1440	109, 101, 92
533×210×82	566	1.8	3.3	5.2	7.0	837	731	1.5	3.0	4.6	6.1	1080	
457×191×98	592	1.8	3.5	5.8	7.6	847	777	1.6	2.9	5.0	7.0	1100	89, 82, 74
457×191×67	405	1.6	3.1	4.9	6.6	636	523	1.4	2.8	4.3	5.8	821	
457×152×82	477	1.3	2.5	4.3	6.3	791	622	1.1	2.4	3.8	5.3	1030	74, 67, 60
457×152×52	301	1.2	2.3	3.7	4.9	564	389	1.1	2.1	3.2	4.3	728	
406×178×74	415	1.6	3.2	5.1	7.3	661	536	1.4	2.8	4.5	6.3	853	67, 60
406×178×54	289	1.5	2.9	4.5	6.2	505	373	1.3	2.6	4.1	5.4	652	
406×140×46	245	1.2	2.3	3.5	4.9	458	316	1.1	2.1	3.2	4.2	591	
406×140×39	198	1.2	2.2	3.3	4.5	413	255	1.0	1.9	3.0	3.9	533	
356×171×67	334	1.6	3.1	5.3	7.7	547	430	1.4	2.8	4.5	6.5	706	57, 51
356×171×45	213	1.5	2.8	4.5	6.1	401	244	1.3	2.4	4.0	5.3	517	
356×127×39	180	1.1	2.0	3.3	4.4	378	232	0.9	1.7	2.9	3.8	488	
356×127×33	148	1.0	2.0	3.0	4.1	339	192	0.9	1.8	2.8	3.6	438	
305×165×54	232	1.6	3.1	5.2	7.8	395	300	1.4	2.8	4.5	6.5	510	46
305×165×40	172	1.5	2.9	4.7	6.5	306	222	1.3	2.6	4.1	5.6	395	
305×127×48	194	1.1	2.3	3.7	5.5	456	251	1.0	2.0	3.2	4.7	588	42
305×127×37	149	1.1	2.1	3.3	4.7	361	192	0.9	1.8	2.9	4.1	466	
305×102×33	132	0.9	1.7	2.7	3.7	341	170	0.8	1.5	2.3	3.3	440	28
305×102×25	92.4	0.8	1.5	2.3	3.2	292	120	0.7	1.3	2.1	2.7	377	
254×146×43	156	1.4	2.8	4.9	7.3	313	202	1.2	2.5	4.2	5.4	404	37
254×146×31	109	1.3	2.5	4.2	5.8	253	125	1.2	2.6	4.1	5.6	327	
254×102×28	97.4	0.9	1.7	2.8	4.0	275	127	0.8	1.6	2.5	3.5	355	25
254×102×22	71.6	0.8	1.6	2.5	3.4	243	93	0.7	1.4	2.3	3.0	314	
203×133×30	86.2	1.3	2.6	4.4	6.6	215	111	1.1	2.4	3.9	5.4	278	
203×133×25	71.2	1.3	2.4	4.1	5.9	194	82	1.1	1.7	2.8	4.0	251	

Universal Columns	GRADE 43						GRADE 50						
D×b×Mass (mm×mm ×Kg/m)	M_{cx} kNm	L_1 m (1.0)	L_2 m (0.75)	L_3 m (0.5)	L_4 m (0.35)	P_v kN	M_{cx} kNm	L_1 m (1.0)	L_2 m (0.75)	L_3 m (0.5)	L_4 m (0.35)	P_v kN	Intermediate masses (kg/m)
356×406×634	3490	8.7	-	-	-	3320	4520	6.8	-	-	-	4410	551, 467, 393,
356×406×235	1240	5.0	12.0	-	-	1120	1620	4.2	16.0	-	-	1460	& 340, 287
356×368×202	1050	4.8	10.5	-	-	1000	1370	3.9	9.0	15.0	-	1300	177, 153
356×368×129	601	4.1	9.8	-	-	605	782	4.8	8.7	14.0	-	788	
305×305×283	1300	4.8	14.0	-	-	1500	1730	4.4	11.5	-	-	2000	240, 198, 158
305×305×97	397	3.2	6.8	12.2	-	503	512	4.0	6.0	10.2	-	649	& 137, 118
254×254×167	641	3.3	10.3	-	-	883	834	3.0	8.7	-	-	1150	132, 107, 89
254×254×73	272	2.3	6.0	11.0	-	360	318	3.4	6.2	10.6	15.0	465	
203×203×86	259	2.7	7.0	14.0	-	459	338	2.2	5.9	12.0	-	598	71, 60, 52
203×203×46	137	2.2	4.8	8.7	13.7	245	159	2.7	5.0	8.2	12.5	316	
152×152×37	85	1.8	4.1	8.1	-	216	110	1.7	3.5	6.8	10.8	279	30
152×152×23	45.4	1.5	3.3	5.6	8.8	153	58.6	2.0	3.5	5.6	8.2	198	

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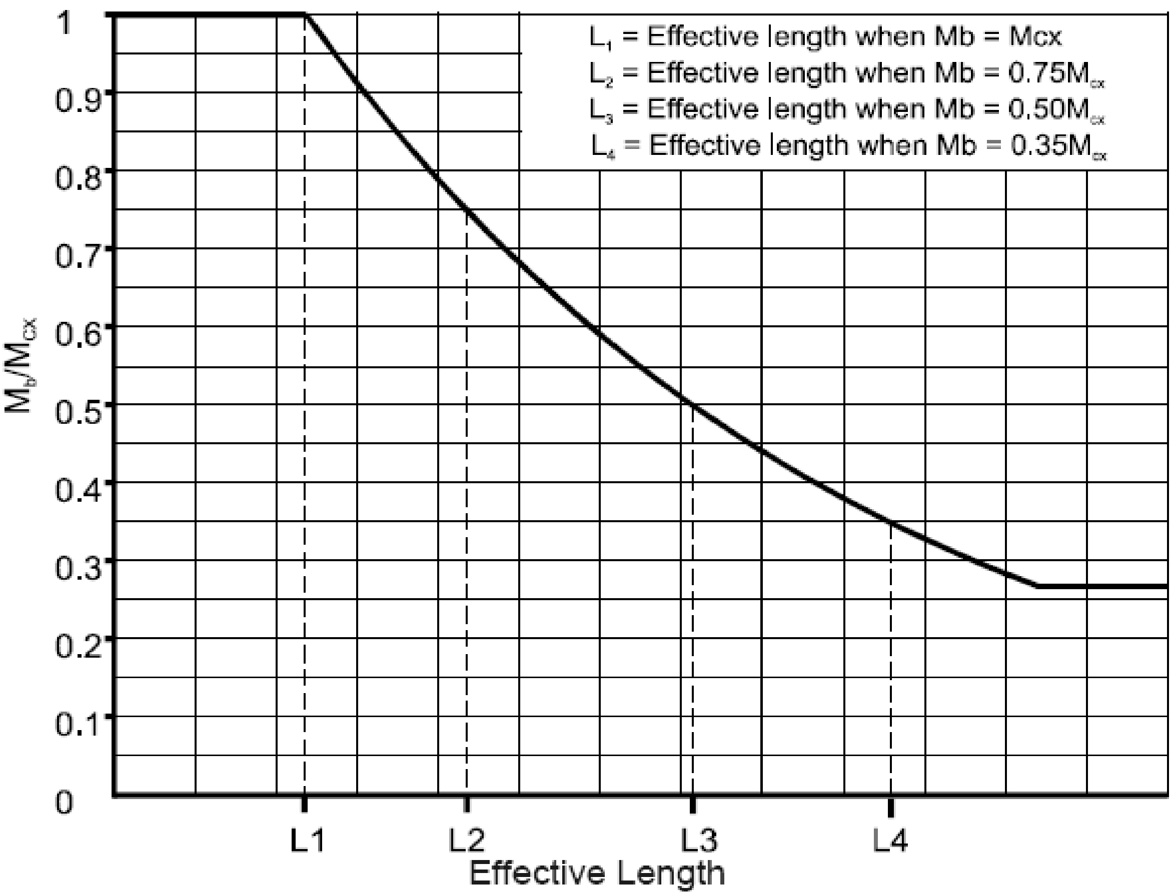
Approximate M_b calculation

Table is to used in conjunction with the table on P. 4/23 to calculate approximate M_b.



Example : 533x210x82UB (p_y = 275 Mpa) with Le compression flange = 6m.

From table L_4 = 7.0m = 0.35M_{cx}

L_3 = 5.2m = 0.50M_{cx}

M_{cx} = 566 kNm

From graph M_b = 0.43M_{cx} (approx.), for Le = 6m.

= 243 kNm

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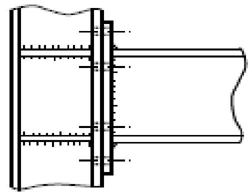
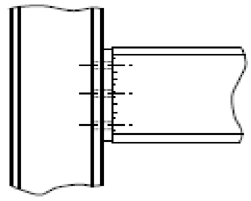
Effective lengths of beam compression flanges

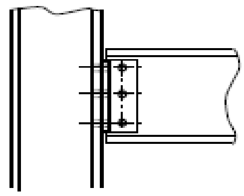
Rotational restraint on plan



1. Flanges fully restrained on plan



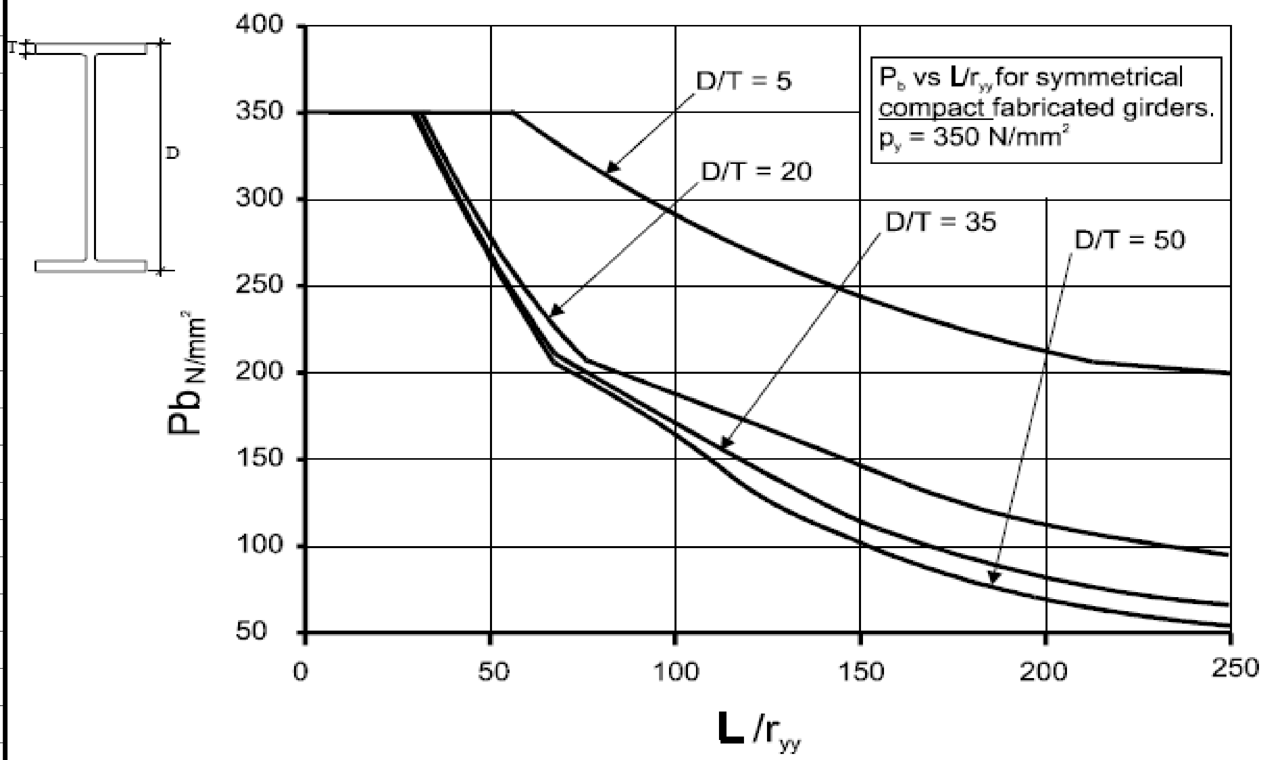
2. Flanges partially restrained on plan



3. Flanges free to rotate on plan

Conditions of restraint at the ends of the beams		Loading conditions	
		Normal	Destabilizing
Compression flange laterally restrained; beam fully restrained against torsion	Both flanges fully restrained against rotation on plan	0.7L	0.85L
	Both flanges partially restrained against rotation on plan	0.85L	1.0L
	Both flanges free to rotate on plan	1.0L	1.2L
Compression flange laterally unrestrained; both flanges free to rotate on plan	Restraint against torsion provided only by positive connection of bottom flange to supports	1.0L+2D	1.2L+2D
	Restraint against torsion provided only by dead bearing of bottom flange on supports.	1.2L+2D	1.4L+2D

Lateral torsional buckling - Stress of fabricated girders



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4.4.5 COLUMNS (AND BEAM COLUMNS)

Local capacity check:

$$\frac{F_c}{A_g p_y} + \frac{M_x}{M_{cx}} + \frac{M_y}{M_{cy}} \le 1$$

P_y = squash load

Buckling check: (minor axis failure)
(Simplified check from from BS5950 4.8.3.3.1)

$$\frac{F_c}{P_c} + \frac{m_x M_x}{p_y Z_x} + \frac{m_y M_y}{p_y Z_y} \le 1$$

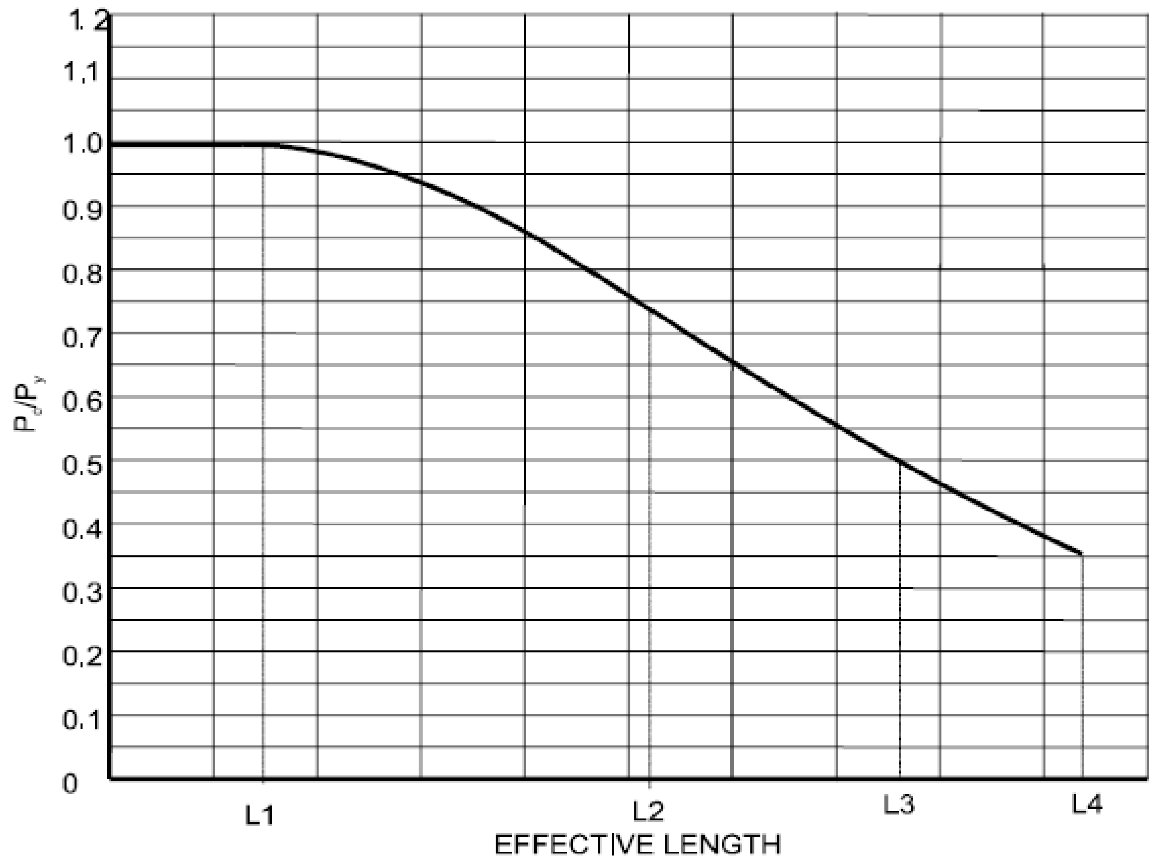
$$\frac{F_c}{P_{cy}} + \frac{m_{LT} M_{LT}}{p_y Z_x} + \frac{m_y M_y}{p_y Z_y} \le 1$$

Where: $m = 0.2 + \frac{0.1M_2 + 0.6M_3 + 0.1M_4}{M_{max}}$ but $m \ge \frac{0.8M_{24}}{M_{max}}$

for m_{LT} see 4.4.4

M_b is obtained from the graph in 4.4.4
P_c is the buckling capacity from table below

Note: For columns in simple construction use m = 1.0; when determining M_b use L = 0.5 H, where H = column height



Note: This graph shows the approximate relationship between axial capacity and effective length. --- see following tables.

L_1 = Effective length when $P_c = P_y$. L_2 = Effective length when $P_c = 0.75P_y$.
L_3 = Effective length when $P_c = 0.50P_y$. L_4 = Effective length when $P_c = 0.35P_y$.

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COMPRESSION

Circular Hollow Sections (CHS)		GRADE 43 (S275)					GRADE 50 (S355)					
Outside diameter (mm)	Thickness (mm)	$P_{c\,max}$ kN	L_{y1} m (1.0)	L_{y2} m (0.75)	L_{y3} m (0.5)	L_{y4} m (0.35)	$P_{c\,max}$ kN	L_{y1} m (1.0)	L_{y2} m (0.75)	L_{y3} m (0.5)	L_{y4} m (0.35)	Intermediate thicknesses * (mm)
88.9	3.2	237	0.4	2.3	3.3	4.1	306	0.4	2.1	3.0	3.7	4.0
	5.0	363	0.4	2.2	3.2	4.0	469	0.4	2.1	2.9	3.6	
114.3	3.6	344	0.6	3.1	4.3	5.3	444	0.6	2.8	3.8	4.8	5.0
	6.3	589	0.6	3.0	4.3	5.2	760	0.6	2.7	3.7	4.6	
139.7	5.0	583	0.7	3.7	5.2	6.4	753	0.7	3.4	4.6	5.8	6.3, 8.0
	10.0	1120	0.7	3.6	5.0	6.2	1440	0.7	3.3	4.5	5.7	
168.3	5.0	707	0.8	4.5	6.3	7.9	912	0.8	4.2	5.6	7.0	6.3, 8.0
	10.0	1370	0.8	4.4	6.1	7.5	1760	0.8	4.0	5.7	6.7	
193.7	5.0	814	1.0	5.2	7.3	9.0	1050	1.0	4.8	6.7	8.0	6.3, 8.0, 10.0
	12.5	1960	0.9	5.0	7.0	8.6	2530	0.9	4.5	6.3	7.7	
219.1	5.0	924	1.1	6.0	8.3	10.0	1190	1.1	5.4	7.3	9.1	6.3, 8.0, 10.0
	12.5	2230	1.1	5.7	8.0	9.9	2880	1.1	5.1	7.1	8.7	
244.5	6.3	1300	1.2	6.7	9.3	11.4	1670	1.2	6.0	8.2	10.1	8.0, 10.0, 12.5
	16.0	3160	1.2	6.5	8.9	11.0	4080	1.2	5.8	7.9	9.7	
273.0	6.3	1450	1.4	7.6	10.3	12.7	1870	1.4	6.8	9.2	11.3	8.0, 10.0, 12.5
	16.0	3550	1.3	7.2	9.9	12.3	4580	1.3	6.5	8.9	10.9	
323.9	6.3	1730	1.7	8.8	12.3	-	2230	1.7	8.0	11.0	13.5	8.0, 10.0, 12.5
	16.0	4260	1.6	8.6	12.0	-	5500	1.6	7.7	10.6	13.0	
355.6	8.0	2400	1.8	9.7	13.5	-	3100	1.8	8.7	12.0	-	10.0, 12.5
	16.0	4700	1.8	9.5	13.1	-	6070	1.8	8.5	11.7	-	

* Only part of the range is given. For the larger sections thicker tubes may be available.

Universal Columns	GRADE 43					GRADE 50				
D×b×Mass (mm×mm×Kg/m)	$P_{c\,max}$ kN	L_{y1} m (1.0)	L_{y2} m (0.75)	L_{y3} m (0.5)	L_{y4} m (0.35)	$P_{c\,max}$ kN	L_{y1} m (1.0)	L_{y2} m (0.75)	L_{y3} m (0.5)	L_{y4} m (0.35)
356×406×634	19800	2.0	5.5	9.2	12.8	26300	1.7	5.1	8.6	11.6
356×406×551	17200	2.0	5.4	9.3	12.7	22800	1.7	4.9	8.6	11.6
356×406×467	15200	1.9	5.3	9.1	12.3	20200	1.7	4.9	8.3	11.0
356×406×393	12800	1.9	5.6	9.5	12.6	17000	1.8	4.8	8.2	10.8
356×406×340	11000	1.9	5.6	9.4	12.5	14700	1.9	4.8	8.1	10.7
356×406×287	9690	1.8	5.9	9.6	12.7	12600	1.7	5.4	8.5	11.2
356×406×235	7950	1.8	5.9	9.6	12.5	10300	1.9	5.4	8.6	11.3
356×368×202	6840	1.8	5.6	9.0	11.8	89000	1.6	5.0	8.2	10.5
356×368×177	5980	1.7	5.7	8.9	11.7	7780	1.7	5.0	8.1	10.5
356×368×153	5180	1.8	5.5	8.9	11.6	6750	1.6	5.0	8.0	10.4
356×368×129	4380	1.9	5.7	8.8	11.5	5700	1.5	4.9	8.0	10.3
305×305×283	9190	1.5	4.6	7.5	9.9	12300	1.3	3.8	6.4	8.7
305×305×240	8090	1.5	4.7	7.7	10.0	10500	1.3	4.2	6.9	8.9
305×305×198	6690	1.5	4.7	7.6	9.8	8710	1.3	4.2	6.8	8.8
305×305×158	5320	1.4	4.7	7.4	9.7	6930	1.3	4.1	6.7	8.7
305×305×137	4620	1.4	4.5	7.3	9.6	6010	1.2	4.1	6.6	8.6
305×305×118	3970	1.4	4.5	7.3	9.6	5160	1.2	4.1	6.6	8.6
305×305×97	3390	1.3	4.4	7.2	9.4	4380	1.1	4.0	6.5	8.4
254×254×167	5630	1.3	3.9	6.3	8.3	7330	1.1	3.6	5.8	7.5
254×254×132	4470	1.2	3.9	6.3	8.3	5820	1.1	3.5	5.7	7.4
254×254×107	3620	1.2	3.8	6.2	8.1	4710	1.1	3.5	5.6	7.3
254×254×89	3010	1.2	3.8	6.2	8.1	3920	1.0	3.5	5.6	7.2
254×254×73	2560	1.1	3.7	6.0	7.9	3300	1.0	3.5	5.5	7.0
203×203×86	2920	0.9	3.1	5.0	6.6	3800	0.9	2.8	4.5	5.8
203×203×71	2410	0.9	3.1	4.9	6.4	3140	0.9	2.7	4.5	5.7
203×203×60	2090	0.9	3.0	4.8	6.3	2700	0.9	2.7	4.4	5.6
203×203×52	1830	0.9	2.9	4.7	6.2	2360	0.8	2.7	4.4	5.6
203×203×46	1620	0.9	2.9	4.7	6.2	2090	0.8	2.7	4.3	5.5
152×152×37	1300	0.7	2.1	3.5	4.7	1680	0.6	2.0	3.3	4.2
152×152×30	1060	0.7	2.2	3.5	4.6	1360	0.6	2.0	3.2	4.2
152×152×23	816	0.7	2.1	3.4	4.5	1050	0.6	2.0	3.1	4.0

NOTE: $L_x \approx 1.15 \left(\frac{I_x}{I_y}\right)^{1/4} L_y$

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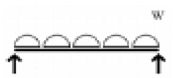
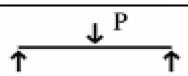
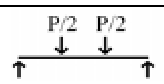
4.4.7 ELEMENT STIFFNESS

Serviceability check: unfactored dead + imposed
unfactored dead + 0.8 × (imposed + wind)

Deflection limits under imposed load:

Element	Limit
• Cantilever	L/180
• Beam supporting plaster or brittle finish	L/360
• Beams supporting masonry	L/500
• Other beams	L/200
• Crane beams	L/500
• Columns	H/300
• Columns in multi-storey construction with movement sensitive cladding.	H/500
Portal frames	
• Lateral at eaves	H/100 - H/300 *
• Vertical at apex	L/250 - L/500 *

* Depends on cladding system

Load case	Minimum I to satisfy deflection limit		
	L/200	L/360	L/500
	1.27 WL²	2.29 WL²	3.18 WL²
	2.03 PL²	3.66 PL²	5.08 PL²
	1.73 PL²	3.12 PL²	4.33 PL²

Note: For castellated beams, assume a 30% increase in deflection due to presence of web openings.
L in metres; W, P in kN; I in cm^4

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