

Calculations Focussing On:

Structural Design of a Small Office Building

Module Code – ACBA6018

Word Count Required – 2000 (+/- 10%)

Word Count: 2200

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Submission Deadline – 5th May 2026

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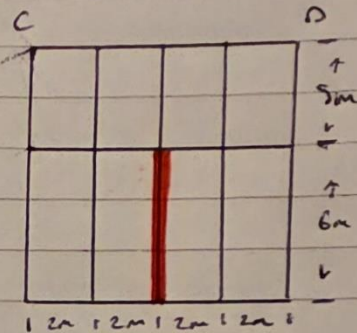
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1. Interior Beam Design S275

Ethan Evans (2211009)

Selection of Beam in S275 Steel (BS5950)

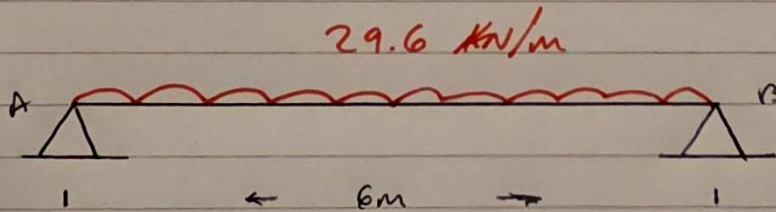
- * Simply Supported
- * Uniformly distributed
- * Dead / Imposed = Dead = 6 kN/m
Imposed = 4 kN/m
- * Fully restrained
- * Universal Beam S275



→ Ultimate load →

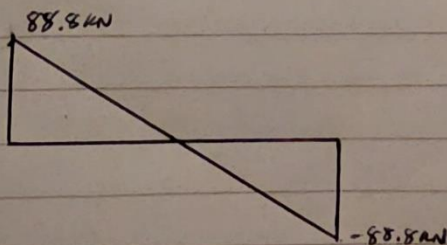
$$\text{Uniform load } w = 1.4g_k + 1.6q_k \\ = 1.4 \times 6 + 1.6 \times 4 = 14.8 \text{ kN/m}$$

Interior beam carrying two panels $\Rightarrow$ Loads doubled
 $14.8 \times 2 = 29.6 \text{ kN/m}$

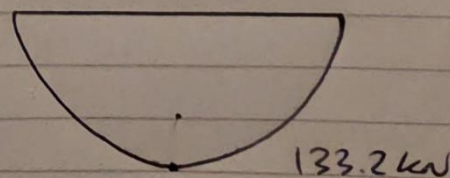


$$\text{Max moment at mid span } = M = \frac{wL^2}{8} = \frac{29.6 \times 8^2}{8} = 133.2 \text{ kN/m}$$

$$\text{Max shear force at Support } \Rightarrow R_A = R_B = \frac{wL}{2} = \frac{29.6 \times 8}{2} = 88.8 \text{ kN/m}$$



Shear force



Bending Moment

Ethan Evans (2211009)

Beam Selection

$$p_y = 275 \text{ mm}^2$$

$$\text{Design} \Rightarrow M \leq M_c \text{ and } M_c = p_y S_x$$

$$\rightarrow M \leq p_y S_x \Rightarrow S_x \geq \frac{M}{p_y} \Rightarrow S_x \geq \frac{133.2 \times 10^6}{275} = 484.3 \text{ cm}^3$$

$$305 \times 127 \times 37 \text{ UB: } S_x = 539 \text{ cm}^3$$

$D = 304.4 \text{ mm}$	$b/T = 5.77$
$B = 123.4 \text{ mm}$	$d/t = 37.4$
$t = 7.1 \text{ mm}$	
$T = 10.7 \text{ mm}$	$S_x = 539 \text{ cm}^3$
$r = 8.9 \text{ mm}$	$Z_x = 471 \text{ cm}^3$
$d = 265.2 \text{ mm}$	

Classification

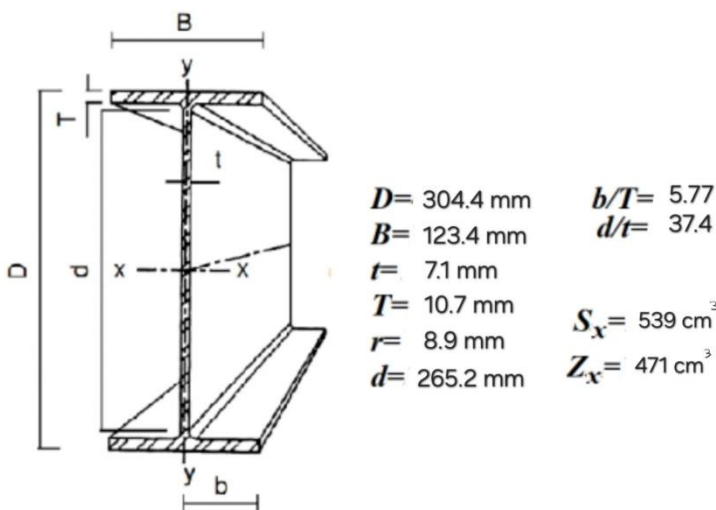
$$\text{flange} = 10.7 \text{ mm} < 16 \text{ mm} \quad p_y = 275 \text{ N/mm}^2$$

Section Classification (Table 11 BS5950)

$$b/T = 5.77 \rightarrow \text{less than } 9 \text{ \& } \epsilon = 9 \quad \text{flange is plastic}$$

$$d/t = 37.4 \rightarrow \text{less than } 80 \quad \epsilon = 80 \quad \text{web is plastic}$$

Section UB 305 x 127 x 37 = Class 1 plastic



Universal Beam 305 x 127 x 37 Interior Spans

Universal beams (UB), Section properties

Dimensions & properties

BS 5950 (BS EN 10365: 2017)

Section designation		Dimensions										Properties					
		Mass per metre	Depth of section	Width of section	Thickness		Root radius	Depth between fillets	Ratios for		Radius of gyration		Elastic modulus		Plastic modulus		Area of section
					Web	Flange			Web	Flange	Axis x-x	Axis y-y	Axis x-x	Axis y-y	Axis x-x	Axis y-y	
					t	T			d / t	b / T	cm	cm	cm ³	cm ³	cm ³	cm ³	
		kg/m	D mm	B mm	t mm	T mm	r mm	d mm			cm	cm	cm ³	cm ³	cm ³	cm ³	A cm ²
305 x 127	x 37	37.0	304.4	123.4	7.1	10.7	8.9	265.2	37.4	5.77	12.3	2.67	471	54.5	539	85.4	47.2

Ethan Evans (2211009)

Shear Strength

$$d/T = 37.4 < 70 \text{ } \epsilon \text{ Shear buckling not considered}$$

Shear capacity of section P_v is $\Rightarrow$

$$P_v = 0.6 p_y A_v = 0.6 p_y t D = 0.6 \times 275 \times 7.1 \times 304.4 = 356 \text{ kN}$$

$$P_v = (88.8) < 0.6 P_v = 0.6 P_v = 0.6 \times 356 = 213.6 \text{ kN}$$

Bending Moment

Moment capacity at section M_c is $\Rightarrow$

$$M_c = p_y S_x = 275 \times 539 \times 10^3 = 148 \text{ kN.m}$$

$$< 1.2 p_y S_x = 1.2 \times 275 \times 471 \times 10^3 = 129 \text{ kN.m}$$

Moment due to ultimate loading = 133.2 kN.m

$$\text{Extra moment due to self weight, } M_{sw} \Rightarrow \frac{1.4(37 \times 10) \times 6^2}{8} = 2.33 \text{ kN.m}$$

$$\text{Total moment} = 133.2 + 2.33 = 135.5 \text{ kN.m}$$

$$135.5 < 148 \text{ kN.m} \quad \checkmark$$

Use Section for all interior beams

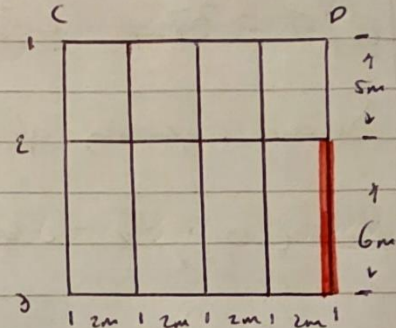


2. Exterior Beam Design S275

Ethan Evans (2211009)

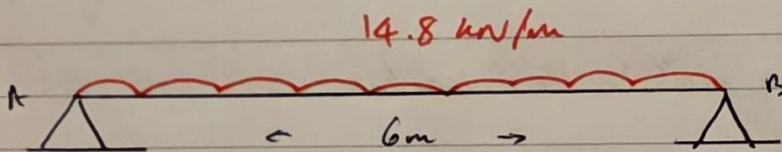
Selection of beam in S275 Steel (BS5950)

- * Simply Supported
- * Uniformly distributed
- * Dead/Imposed load = Dead = 6 kN/m
Imposed = 4 kN/m
- * fully restrained
- * Universal beam S275



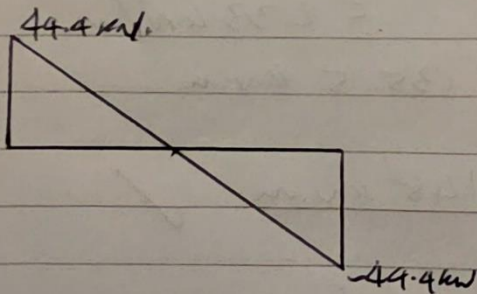
→ Ultimate load →

$$\text{Uniform Load } w = 1.4g_k + 1.6q_k = 1.4 \times 6 + 1.6 \times 4 = 14.8 \text{ kN/m}$$

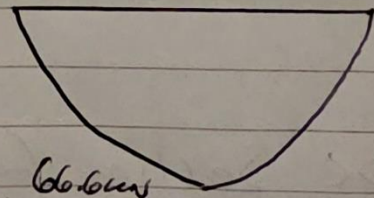


$$\text{Max moment at mid span} = M = \frac{wL^2}{8} = \frac{14.8 \times 6^2}{8} = 66.6 \text{ kNm}$$

$$\text{Max shear force at support} \Rightarrow R_A = R_B = \frac{wL}{2} = \frac{14.8 \times 6}{2} = 44.4 \text{ kN}$$



Shear force



Bending Moment

Selection of Beam

$$p_y = 275 \text{ N/mm}^2$$

Design $\rightarrow M \leq M_c$ and $M_c = p_y S_x$

$$M \leq p_y S_x \rightarrow S_x \geq \frac{M}{p_y} \rightarrow S_x \geq \frac{66.6 \times 10^6}{275} = 242 \text{ cm}^3$$

Steel Selection

$$203 \times 133 \times 25 \text{ UB: } S_x = 258 \text{ cm}^3 \rightarrow$$

$D = 203.2 \text{ mm}$	$b/T = 8.54$
$B = 133.2 \text{ mm}$	$d/t = 30.2$
$t = 5.7 \text{ mm}$	
$T = 7.8 \text{ mm}$	$S_x = 258 \text{ cm}^3$
$r = 7.6 \text{ mm}$	$Z_x = 230 \text{ cm}^3$
$d = 172.4 \text{ mm}$	

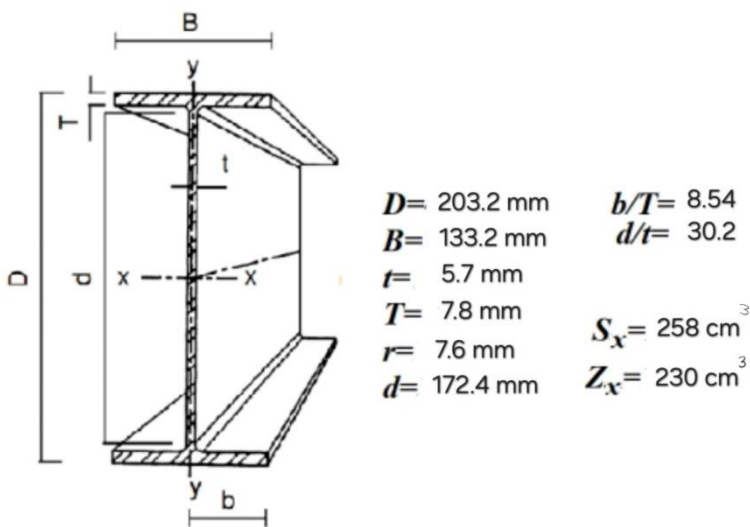
Section Classification (Table 11 BS5950)

flange $= 7.8 \text{ mm} < 16 \text{ mm}$ $p_y = 275 \text{ N/mm}^2$

$b/T = 8.54 \rightarrow$ less than 9 $\epsilon = 9 \rightarrow$ flange is plastic

$d/t = 30.2 \rightarrow$ less than 80 $\epsilon = 80 \rightarrow$ web is plastic

Section UB $203 \times 133 \times 25$ is class 1 plastic



Universal Beam $203 \times 133 \times 25$ Exterior Spans

Universal beams (UB), Section properties

Dimensions & properties

BS 5950 (BS EN 10365: 2017)

Section designation

Section designation		Dimensions										Properties							
		Mass per metre	Depth of section	Width of section	Thickness		Root radius	Depth between fillets	Ratios for		Radius of gyration		Elastic modulus		Plastic modulus		Area of section		
					Web	Flange			Web	Flange	Axis x-x	Axis y-y	Axis x-x	Axis y-y	Axis x-x	Axis y-y			
					t	T			d / t	b / T	cm	cm	cm ³	cm ³	cm ³	cm ³			
		kg/m	D	B	t	T	r	d			cm	cm	cm ³	cm ³	cm ³	cm ³	A		
203 x 133	x 25	25.1	203.2	133.2	5.7	7.8	7.6	172.4	30.2	8.54	8.56	3.10	230	46.2	258	70.9	32.0		

Ethan Evans (2211009)

Shear Strength

$d/T = 30.2 < 70 \epsilon$ Shear buckling not considered

Shear Capacity of Section P_v is $\rightarrow$

$$P_v = 0.6 p_y A_v = 0.6 p_y t D = 0.6 \times 275 \times 5.7 \times 203.2 = 191 \text{ kN}$$

$$F_v = (44.4) < 0.6 P_v = 0.6 \times 191 = 114.6 \text{ kN}$$

Bending Moment

Moment Capacity at Section M_c is $\rightarrow$

$$M_c = p_y S_x = 275 \times 258 \times 10^3 = 70.95 \text{ kNm}$$

$$< 1.2 p_y Z_{xc} = 1.2 \times 275 \times 230 \times 10^3 = 75.9 \text{ kNm}$$

Moment due to ultimate loading = 66.6 kNm

Extra moment due to self weight, $M_{sw} = \frac{1.4(25.1 \times 10) \times 5^2}{8} = 1.09 \text{ kNm}$

Total Moment, $M_t = 66.6 + 1.09 = 67.69 \text{ kNm}$

$$67.69 < 70.95 \text{ kNm}$$

Use beam for edge/axial beams

3. Interior & Exterior Girder Design S275

Ethan Evans (2211009)

Selection of Girder in S275 Steel (B55482)

* Simply Supported

* Concentrated loads from 3 beams

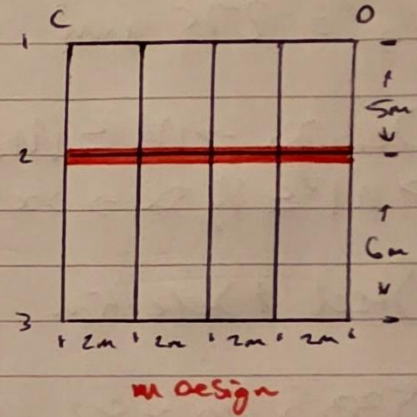
* Dead/Imposed = Dead = 6 kN/m

Imposed = 4 kN/m

* Universal beam S275

→ Point load → 88.8 from either side

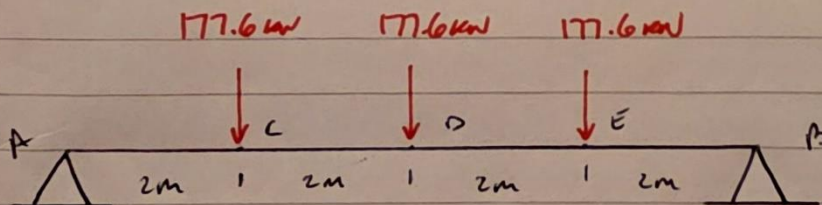
$$88.8 \times 2 = 177.6 \text{ kN}$$



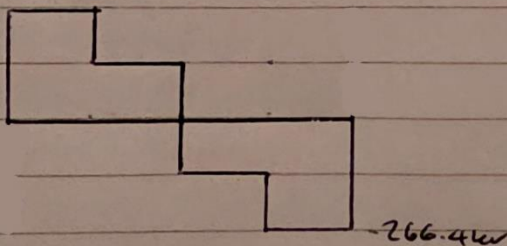
Max moment at mid span

$$M = \frac{3PL}{8} = \frac{3 \times (177.6 \times 8)}{8} = 532.8 \text{ kNm}$$

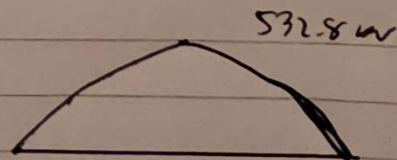
$$\text{Max Shear force at Support} \rightarrow R_A = R_B = \frac{3P}{2} = \frac{3 \times 177.6}{2} = 266.4 \text{ kN}$$



266.4 kN



Shear force



Bending Moment

Ethan Evans (2211009)

Selection of Girder

$$p_y = 275 \text{ N/mm}^2$$

Design $\rightarrow M \leq M_c$ and $M_c = p_y S_x$

$$M \leq p_y S_x \rightarrow S_x \geq \frac{M}{p_y} \rightarrow S_x \geq \frac{532.8 \times 10^6}{275} = 1934 \text{ cm}^3$$

Steel Selection

$$533 \times 210 \times 82 \text{ UB } S_x = 2060 \text{ cm}^3$$

Steel Selection

$D = 528.3 \text{ mm}$	$b/t = 7.91$
$B = 208.8 \text{ mm}$	$d/t = 49.6$
$t = 9.6 \text{ mm}$	
$T = 13.2 \text{ mm}$	$S_x = 2060 \text{ cm}^3$
$r = 12.7 \text{ mm}$	$Z_x = 1800 \text{ cm}^3$
$d = 476.5 \text{ mm}$	

$$533 \times 210 \times 82 \text{ UB } S_x = 2060 \text{ cm}^3$$

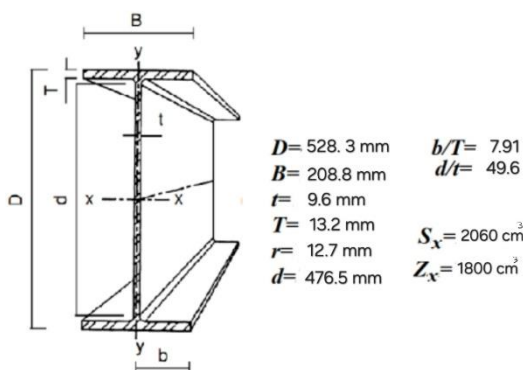
Classification (Table 11 (BS5950))

$$\text{flange} = 13.2 < 16 \text{ mm} \quad p_y = 275 \text{ N/mm}^2$$

$$b/t = 7.91 \rightarrow \text{less than } 9 \quad \epsilon = 9 \quad \text{flange is plastic}$$

$$d/t = 49.6 \rightarrow \text{less than } 80 \quad \epsilon = 80 \quad \text{web is plastic}$$

Section UB 533 x 210 x 82 is class 1 plastic



Universal Beam 533 x 210 x 82 Interior & Exterior Girders

Universal beams (UB), Section properties

Dimensions & properties

BS 5950 (BS EN 10365: 2017)

Section designation		Dimensions										Properties							
		Mass per metre	Depth of section	Width of section	Thickness		Root radius	Depth between fillets	Ratios for		Radius of gyration		Elastic modulus		Plastic modulus		Area of section		
					Web	Flange			Web	Flange	Axis x-x	Axis y-y	Axis x-x	Axis y-y	Axis x-x	Axis y-y			
					<i>t</i>	<i>T</i>			<i>d</i> / <i>t</i>	<i>b</i> / <i>T</i>	cm	cm	cm ³	cm ³	cm ³	cm ³			
		<i>D</i>	<i>B</i>	<i>t</i>	<i>T</i>	<i>r</i>	<i>d</i>										<i>A</i>		
		kg/m	mm	mm	mm	mm	mm	mm			cm	cm	cm ³	cm ³	cm ³	cm ³	cm ²		
533 x 210	x 82	82.2	528.3	208.8	9.6	13.2	12.7	476.5	49.6	7.91	21.3	4.38	1800	192	2060	300	105		

Ethan Evans (2211009)

Shear Strength

$$d/T = 49.6 < 70 \text{ } \epsilon \text{ Shear buckling not required}$$

Shear capacity of section P_v is $\rightarrow$

$$P_v = 0.6 p_y A_v = 0.6 p_y t_w = 0.6 \times 275 \times 9.6 \times 528.3 = 836.8 \text{ kN}$$

$$F_v = (266.4) \rightarrow 0.6 \times 1800 = 1080 \text{ kN}$$

Bending Moment

Moment capacity at section M_c is $\rightarrow$

$$M_c = p_y S_x = 275 \times 2060 \times 10^3 = 566.5 \text{ kN.m}$$

$$< 1.2 p_y Z_x = 1.2 \times 275 \times 1800 \times 10^3 = 594 \text{ kN.m}$$

$$\text{Moment due to Self weight, } M_{sw} = \frac{1.4 \times (82.2 \times 10)^8^2}{8} = 9.2 \text{ kN}$$

$$\text{Total moment, } M_t = 532.8 + 9.2 = 538 \text{ kN.m}$$

$$538 < 594 \text{ kN.m}$$

Girder to be used for internal girders + Exterior

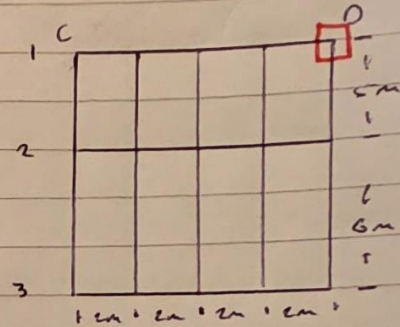


4. Corner Column Design S275

Ethan Evans (2211009)

Design of a Steel Column (BS5950) (corner)

- * Steel column S275
- * Restrainted at bottom / pinned at top
- * Ground floor design
- * Connecting beam / girder calculated

Section Classification

254 x 254 x 89 UC

in corner column design

Thickness $T = 17.3 \text{ mm} > 16 \text{ mm}$ $p_y = 265 \text{ N/mm}^2$ $\epsilon = 1.5$
 flange $b/T = 6.38 \text{ mm} < 9$ $\epsilon = 9$ flange is plastic
 $d/T = 19.4 < \frac{80\epsilon}{1 + \epsilon}$ $r_1 = \frac{p_c}{d \cdot p_{yw}}$ but $-1 < r_1 < 1$ p_c (axial compression)

$$p_c = A + B \text{ reactions} \Rightarrow \frac{1554 \times 10^3}{200.3 \times 10 \times 265} = 2.8, r_1 = 1$$

$$d/T = 19.4 < \frac{80\epsilon}{1 + \epsilon} = d/T = 19.4 < \frac{80 \times 1}{1 + 1} = 40 \rightarrow \text{web is plastic}$$

Section UC 254 x 254 x 89 is plastic

Design loading + momentsUltimate reaction from beam A, $R_A = 266.4 \text{ kN}$ Ultimate reaction from beam B, $R_B = 44.4 \text{ kN}$

$$\text{Column Self weight} = \text{mass} \times g \times \text{height} = 88.8 \times 10 \times 4 \times 10^{-3} = 3.5 \text{ kN}$$

$$\text{Ultimate Axial load, } p_c = R_A + R_B + C_{sw} = 310.8 + 3.5 = 314.3 \text{ kN} \\ \times 5 \\ = \underline{\underline{1571.5 \text{ kN}}}$$

Universal columns (UC), Section properties

Dimensions & properties

Eurocode 3 (in accordance with the UK National Annex) (BS EN 10365: 2017)

Section designation		Dimensions										Properties					
		Mass per metre	Depth of section	Width of section	Thickness		Root radius	Depth between fillets	Ratios for local buckling		Radius of gyration		Elastic modulus		Plastic modulus		Area of section
			<i>D</i>	<i>B</i>	Web <i>t</i>	Flange <i>T</i>	<i>r</i>	<i>d</i>	Web <i>d</i> / <i>T</i>	Flange <i>b</i> / <i>T</i>	Axis x-x	Axis y-y	Axis y-y	Axis z-z	Axis x-y	Axis z-z	<i>A</i>
		kg/m	mm	mm	mm	mm	mm	mm			cm	cm	cm ³	cm ³	cm ³	cm ³	cm ²
254 x 254	x 89	88.9	260.3	256.3	10.3	17.3	12.7	200.3	19.4	6.38	11.2	6.55	1100	379	1220	575	113

+ These sections are in addition to the range of BS EN 10365 sections

Ethan Evans (2211009)

Load Eccentricities

Beam A $\Rightarrow e_x = D/2 + 100 = \frac{260.3}{2} + 100 \Rightarrow 230.15 \text{ mm}$

Beam B $\Rightarrow e_y = B/2 + 100 = \frac{256.3}{2} + 100 \Rightarrow 105.15 \text{ mm}$

Moment

Due to beam A, $M_x = R_A + e_x = 266.4 \times 230.15 \times 10^{-3} = 61.31 \text{ kN}$

Beam B $= M_y = R_B + e_y = 44.4 \times 105.15 \times 10^{-3} = 4.66 \text{ kN}$

Effective length

$L_e = 1 \times 4 = 4 \text{ m}$

Compressive Strength

Slenderness $\Rightarrow \lambda_{xc} = L_e / r_x = \frac{4 \times 10^3}{11.2 \times 10} = 35.71$

$\lambda_y = L_e / r_y = \frac{4 \times 10^3}{6.55 \times 10} = 61.06$

About x-x axis, Table 24(b)

$\rho_{cx} = \lambda = 40 \Rightarrow S265 = 241 \text{ } S275 = 250$

y-y axis Table 24(c)

$\rho_{cy} = \lambda = 62 \Rightarrow S265 = 191 \text{ } S275 = 197$

$\rho_c = 191 \text{ N/mm}^2$

$P_c = A_g \rho_c = \frac{113 \times 10^2 \times 191 \times 10^3}{6.55 \times 10} = \frac{2158.3 \text{ kN}}{6.55 \times 10}$

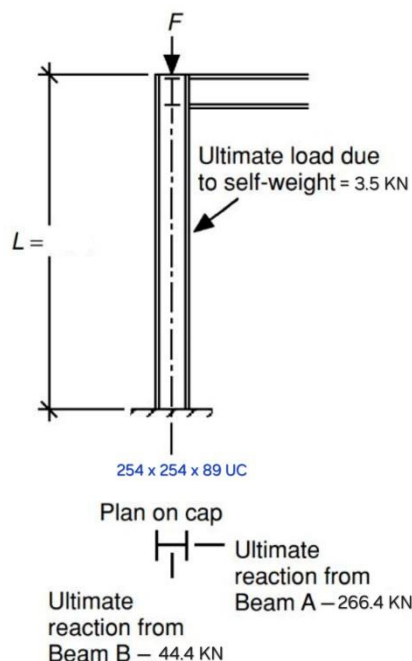


Table 4.13 Compressive strength, p_c (N/mm²) with $\lambda < 110$ for strut curve b (Table 24(b), BS 5950)

λ	Steel grade and design strength p_y (N/mm ²)										
	S275					S355					
	235	245	255	265	275	315	325	335	345	355	400
15	235	245	255	265	275	315	325	335	345	355	399
20	234	243	253	263	272	310	320	330	339	349	391
25	229	239	248	258	267	304	314	323	332	342	384
30	225	234	243	253	262	298	307	316	325	335	375
35	220	229	238	247	256	291	300	309	318	327	366
40	216	224	233	241	250	284	293	301	310	318	355
42	213	222	231	239	248	281	289	298	306	314	351

Design in structural steelwork to BS 5950

Table 4.14 Compressive strength, p_c (N/mm²) for strut curve c (Table 24(c), BS 5950)

λ	Steel grade and design strength p_y (N/mm ²)											
	S275					S355					S46	
	235	245	255	265	275	315	325	335	345	355	400	410
15	235	245	255	265	275	315	325	335	345	355	398	408
20	233	242	252	261	271	308	317	326	336	345	387	396
25	226	235	245	254	263	299	308	317	326	335	375	384
30	220	228	237	246	255	289	298	307	315	324	363	371
35	213	221	230	238	247	280	288	296	305	313	349	357
40	206	214	222	230	238	270	278	285	293	301	335	343
42	203	211	219	227	235	266	273	281	288	296	329	337
44	200	208	216	224	231	261	269	276	284	291	323	330
46	197	205	213	220	228	257	264	271	279	286	317	324
48	195	202	209	217	224	253	260	267	274	280	311	317
50	192	199	206	213	220	248	255	262	268	275	304	310
52	189	196	203	210	217	244	250	257	263	270	297	303
54	186	193	199	206	213	239	245	252	258	264	291	296
56	183	189	196	202	209	234	240	246	252	258	284	289
58	179	186	192	199	205	229	235	241	247	252	277	282
60	176	183	189	195	201	225	230	236	241	247	270	274
62	173	179	185	191	197	220	225	230	236	241	262	267
64	170	176	182	188	193	215	220	225	230	235	255	260

Ethan Evans (2211009)

Buckling Resistance

$$\lambda_{LT} = 0.5 L/r_y = \frac{0.5 \times 4 \times 10^3}{6.55 \times 10} = 30.53$$

Table 16, BS 5950

$$\rho_b = \lambda_{LT} = 35 \rightarrow 5265 = 268$$

Buckling resistance moment capacity of column, M_{bs}

$$M_{bs} = \rho_b S_x = 265 \times 1220 \times 10^{-3} = 323.3 \text{ kNm}$$

Buckling

$$\frac{f_c}{p_c} + \frac{m_x}{M_{bs}} + \frac{m_y}{p_y z_y} \leq 1.0$$

Substitute

$$\frac{1571.5}{2158.3} + \frac{61.31}{323.3} + \frac{4.66}{265 \times 379 \times 10^{-3}} = 1.0$$

Section Satisfactory

5. Edge Column Design S275

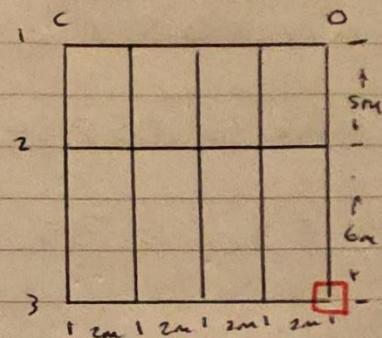
Ethan Evans (2211009)

Design of a Steel Column (B55480) (Edge)

- * Steel column S275
- * Restrained at bottom / pinned at top
- * Ground floor design
- * Connecting beam / girder calculated

Section Classification

254 x 254 x 89 UC



all edge column design

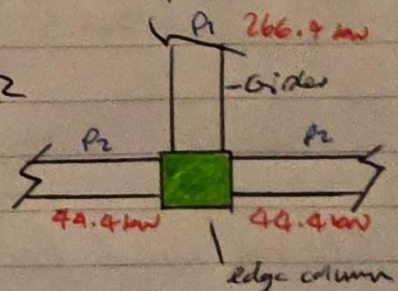
Thickness $T = 17.3 \text{ mm} > 16 \text{ mm}$ $p_1 = 265 \text{ N/mm}^2$ $\epsilon = 1.5$
 flange $b/T = 6.38 \text{ mm} < 9$ $\epsilon = 9$ flange is plastic
 $d/T = 19.4 < \frac{80\epsilon}{1+\epsilon}$ $r_1 = \frac{f_c}{d+T}$ $b/d - 1 < r_1 < r_2$ f_c axial compression

$$f_c = p_1 + p_2 + p_2 \times 5 = 355.2 \times 5 = 1776$$

$$f_c = \frac{1776 \times 10^3}{200.3 \times 10.3 \times 265} = 3.2 > 1 \quad r_1 = 1$$

$$d/T = 19.4 < \frac{80\epsilon}{1+\epsilon} = d/T = 19.4 < \frac{80 \times 1}{1+1} = 40 \rightarrow \text{web is plastic}$$

Section UC 254 x 254 x 89 is plastic

Design loading + momentsUltimate reactions from $p_1 = 266.4 \text{ kN}$ Ultimate reactions from $p_2 = 44.4 + 44.4 \text{ kN}$

$$\text{Column self weight} = \text{mass} \times g \times \text{height} = \frac{88.1 \times 10 \times 4 \times 10^{-3}}{1000} = 3.5 \text{ kN}$$

$$\text{Ultimate Axial load, } f_c = R_a = R_s + f_{sw} = 355.7 \times 5 = 1778.5 \text{ kN}$$

Ethan Evans (2211009)

Load Eccentricities

$$\text{Beam A} \rightarrow e_x = 0/2 + 100 = p_1 \left(100 + \frac{p_2}{2} \right) = 230.15 \text{ mm}$$

$$\text{Beam B} \rightarrow e_y = t/2 + 100 = p_2 \left(100 + \frac{t}{2} \right) = 0.0$$

Moment

$$\text{Due to beam A, } M_x = R_A + e_x = 266.4 \times 230.15 \times 10^{-3} = 61.31 \text{ kN}$$

Effective length

$$L_e = L_c \rightarrow 1 \times 4 = 4 \text{ m}$$

Compressive Strength

$$\text{Slenderness} \rightarrow A_x = L_e / r_x = \frac{4 \times 10^3}{11.2 \times 10} = 35.71$$

$$A_y = L_e / r_y = \frac{4 \times 10^3}{6.55 \times 10} = 61.06$$

about x-x axis Table 24(B)

$$p_x = \lambda = 40 \rightarrow S265 = 241 \quad S275 = 250$$

y-y axis

$$p_y = \lambda \rightarrow 62 \quad S265 = 191 \quad S275 = 192$$

$$p_c = 191 \text{ N/mm}^2$$

$$P_c = A_g p_c = \frac{113 \times 10^2 \times 191 \times 10^3}{6.55 \times 10} = 2158.3 \text{ kN}$$

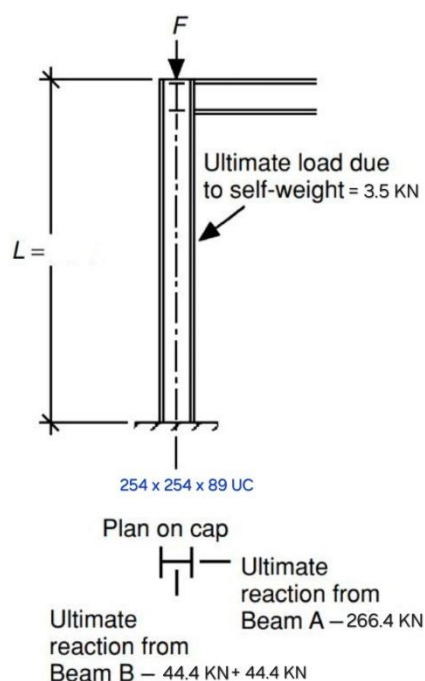


Table 4.13 Compressive strength, p_c (N/mm²) with $\lambda < 110$ for strut curve b (Table 24(b), BS 5950)

λ	Steel grade and design strength p_y (N/mm ²)										
	S275					S355					
	235	245	255	265	275	315	325	335	345	355	400
15	235	245	255	265	275	315	325	335	345	355	399
20	234	243	253	263	272	310	320	330	339	349	391
25	229	239	248	258	267	304	314	323	332	342	384
30	225	234	243	253	262	298	307	316	325	335	375
35	220	229	238	247	256	291	300	309	318	327	366
40	216	224	233	241	250	284	293	301	310	318	355
42	213	222	231	239	248	281	289	298	306	314	351

Design in structural steelwork to BS 5950

Table 4.14 Compressive strength, p_c (N/mm²) for strut curve c (Table 24(c), BS 5950)

λ	Steel grade and design strength p_y (N/mm ²)											
	S275					S355					S46	
	235	245	255	265	275	315	325	335	345	355	400	410
15	235	245	255	265	275	315	325	335	345	355	398	408
20	233	242	252	261	271	308	317	326	336	345	387	396
25	226	235	245	254	263	299	308	317	326	335	375	384
30	220	228	237	246	255	289	298	307	315	324	363	371
35	213	221	230	238	247	280	288	296	305	313	349	357
40	206	214	222	230	238	270	278	285	293	301	335	343
42	203	211	219	227	235	266	273	281	288	296	329	337
44	200	208	216	224	231	261	269	276	284	291	323	330
46	197	205	213	220	228	257	264	271	279	286	317	324
48	195	202	209	217	224	253	260	267	274	280	311	317
50	192	199	206	213	220	248	255	262	268	275	304	310
52	189	196	203	210	217	244	250	257	263	270	297	303
54	186	193	199	206	213	239	245	252	258	264	291	296
56	183	189	196	202	209	234	240	246	252	258	284	289
58	179	186	192	199	205	229	235	241	247	252	277	282
60	176	183	189	195	201	225	230	236	241	247	270	274
62	173	179	185	191	197	220	225	230	236	241	262	267
64	170	176	182	188	193	215	220	225	230	235	255	260

Ethan Evans (2211009)

Buckling Resistance

$$\lambda_{LT} = 0.5 \quad L/r_y = \frac{0.5 \times 4 \times 10^3}{6.55 \times 10} = 30.53$$

Table 16 BS5950

$$P_b = \lambda_{LT} = 35 \rightarrow S265 = 265$$

Buckling resistance moment capacity of column, M_{bs}

$$M_{bs} = p_y S_x = 265 \times 1220 \times 10^{-3} = 323.3 \text{ kN.m}$$

Buckling

$$\frac{f_c}{p_c} + \frac{m_x}{m_{bs}} + \frac{m_y}{p_{zy}} \leq 1.0$$

Substitute

$$\frac{1793.5}{2158.3} + \frac{61.31}{323.3} + \frac{0}{265 \times 371 \times 10^{-3}} = 1.02$$

Section to be used for all columns

$$\underline{\underline{254 \times 254 \times 89 \text{ UC}}}$$

6. Design of a Beam - Column End Plate

Ethan Evans (2211009)

Design of beam-column end plate (BS5950)

* Steel S275

* Bolts are 20mm diameter, grade 8.8

→ 254 x 254 x 89 UC

→ S33 x 210 x 82 UB

→ Bolt Spacing + Edge/end distances

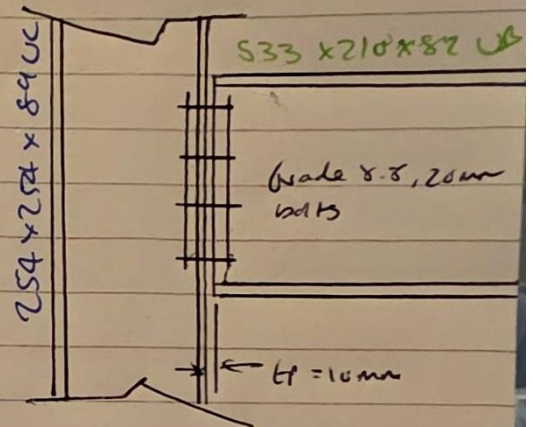
* Bolt diameter, $d_b = 20\text{mm}$

* Hole diameter, $d_h = 20 + 2 = 22\text{mm}$

* Thickness end plate, $t_p = 10\text{mm}$

* Length end plate, $l_p = 460\text{mm}$

* Grade S275 Steel, $p_y = 275\text{N/mm}^2$ $\epsilon = 1$



Conditions

* Pitch of bolt, $p = 120\text{mm}$, $2.5d_b (=50\text{mm}) \leq p \leq 14t (=140\text{mm})$

* Edge + end distance, $e_1 + e_2 = 35 + 50\text{mm}$, $1.4d_b (=30.8\text{mm}) \leq e_1 + e_2 \leq 11t_p (=110\text{mm})$ ✓

Bolt Group Strength

Shear → 8 bolts, 20mm diameter, grade 8.8, bolts single shear

Single shear value 20mm diameter, $P_s = 91.9\text{kN}$

Shear capacity of bolt group is $8P_s = 8 \times 91.9 = 735\text{kN}$

Ethan Evans (2211009)

Bearing Capacity

(minimum of $p_{bb} + p_{bs}$) for 20mm diameter bolt and 10mm plate thickness, (thickness of angle cleat, $t_v = 10\text{mm}$)
 $p_{bs} = 92\text{ kN}$

Bearing Capacity of connection is $8 p_{bs} = 8 \times 92 = 736\text{ kN}$

End plate Shear Strength

Shear capacity of end plate, p_v is (clause 4.2.3, BS5950)

$$p_v = 0.6 p_y A_v$$

$$A_v = 0.9 A_n$$

$$A_n = \text{Net area}$$

$$A_n = (L_p - 4d_n) t_p = 10 \times (460 - 4 \times 22) = 3720\text{ mm}^2$$

$$A_v = 0.9 A_n = 0.9 \times 3720 = 3348\text{ mm}^2$$

Shear capacity assuming single plane failure, p_{vp} is

$$p_{vp} = 0.6 p_y A_v = 0.6 \times 275 \times 3348 = 552.4\text{ kN}$$

Two failure planes

$$2 p_{vp} = 2 \times 552.4 = 1104.8\text{ kN}$$

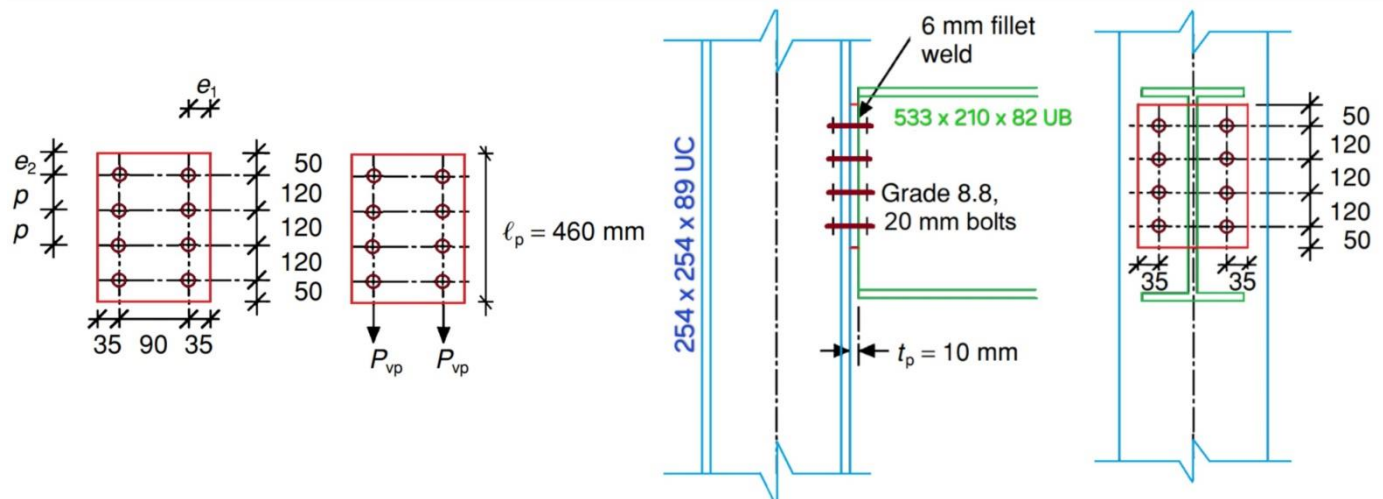
Weld Strength $\rightarrow$ 6mm fillet provided

Design Strength, $p_w = 220\text{ N/mm}^2 \rightarrow$ electrode strength $= 42\text{ N/mm}^2$

$$\text{Throat size, } a = 0.7s = 0.7 \times 6 = 4.2\text{ mm}$$

Effective length of weld, L_w

$$V_w = p_w a L_w = 220 \times 4.2 \times 896 = 828\text{ kN}$$



Ethan Evans (221009)

Shear Strength of beam web at end plate

$$f_{ub} = 0.6 p_y A_v$$

$$A_v = 0.9 t_w L_p = 0.9 \times 9.6 \times 460 = 3974.4 \text{ mm}^2$$

$$f_{ub} = 0.6 p_y A_v = 0.6 \times 275 \times 3974.4 = 655.7 \text{ kN}$$

Summary

- Shear capacity of bolt group = 735 kN
- Bearing capacity of connection is = 736 kN
- Shear capacity of end plate (2 failure planes) = 1104.8 kN
- Shear strength of weld is = 828 kN
- Shear strength of beam web at end plate = 655.7 kN

Strength of connection is controlled by shear strength of beam web and is equal to 655.7 kN

End plate to be used for all connections.

7. Design of a Column Base Plate

Ethan Evans (2211009)

Design of a Steel column baseplate (B55750)

- * Axially Loaded
- * Supported on concrete strength, $f_{cu} = 25 \text{ N/mm}^2$
- * Grade S275 Steel
- * $254 \times 254 \times 89 \text{ UC}$

$$\text{Axial load} = 1571 \text{ kN}$$

$$T = 17.3 \text{ mm}$$

$$E = 10.3 \text{ mm}$$

$$D = 260.3 \text{ mm}$$

$$B = 256.3 \text{ mm}$$

$$\Rightarrow \text{Effective Area, } A_{eff} \rightarrow \frac{\text{Axial Load}}{\text{Bearing Strength}}$$

$$1.6 \times 1571 = 2513.6$$

$$= \frac{2513.6 \times 10^3}{0.6 \times 25}$$

$$= 1.67 \times 10^5 \text{ mm}^2$$

$$\text{Effective Area } \Rightarrow A_{eff} = 4c^2 + (2D + 4B - 2T)c + (2BT + 2E - 2TE)$$

$$\text{Substituting } \Rightarrow A_{eff} = 4c^2 + [(2 \times 260.3) + (4 \times 256.3) - (2 \times 17.3)]c + [(2 \times 256.3 \times 17.3) + (260.3 \times 10.3) - (2 \times 17.3 \times 10.3)]$$

$$A_{eff} = 4c^2 + 1525.2c + 11192.69$$

$$\text{Sub } \rightarrow 167573 = 4c^2 + 1525.2c + 11192.69$$

$$\text{Rearrange } \rightarrow 4c^2 + 1525.2c - 156380.31 = 0$$

$$c = \frac{-1525.2 \pm \sqrt{1525.2^2 + 4(4)(156380.31)}}{8}$$

$$= \frac{-1525.2 \pm \sqrt{2326235.04 + 2502084.96}}{8}$$

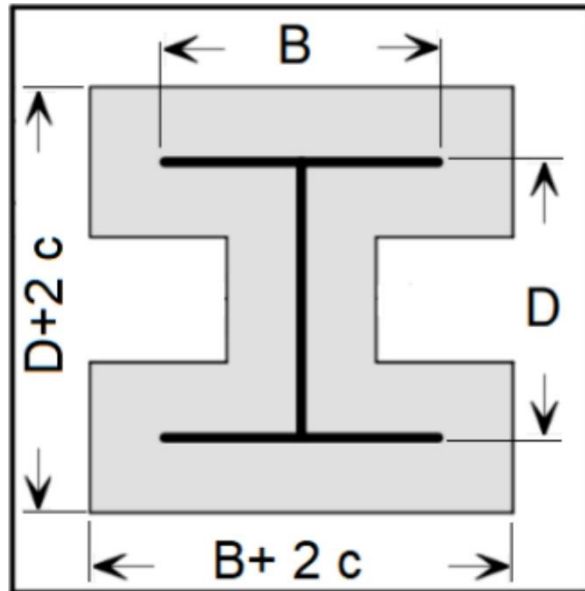
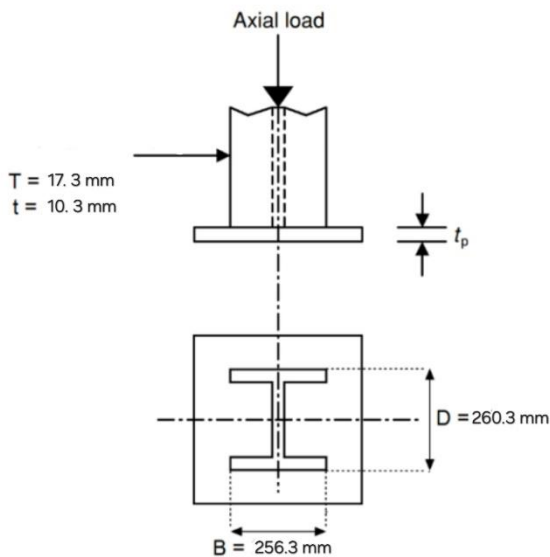
$$= \frac{-1525.2 \pm \sqrt{4828319.99}}{8}$$

$$\sqrt{4828319.99} = 2196.9$$

$$c = \frac{-1525.2 + 2196.2}{8} = 83.9 \text{ mm}$$

$$c = \frac{-1525.2 - 2196.2}{8} = 465.2 \text{ mm}$$

$$c = 84 \text{ mm}$$



Ethan Evans (2211009)

Baseplate Specs

Minimum length: $D + 2c = 260.3 + 2(84) = 428.3 \text{ mm}$

Minimum width: $B + 2c = 256.3 + 2(84) = 424.3 \text{ mm}$

Provide $500 \times 500 = 2.5 \times 10^5 > A_{eff} = 1.67 \times 10^5$ ✓

Thickness

Assuming thickness, $t_p: 16 < t_p \leq 40$

Design Strength: $p_{y1} = 265 \text{ N/mm}^2$

Actual thickness, t_p given by

$$t_p = c \left(\sqrt{\frac{3w}{p_{y1}}} \right)$$

$$t_p = 84 \left(\sqrt{\frac{3 \times 0.5 \times 25}{265}} \right) = 31.5 \text{ mm} > \text{column thickness, } T (= 17.3 \text{ mm}) \quad \checkmark$$

Use $500 \times 500 \times 38$ or 40 mm thick baseplate
in S275 Steel

GRADE 8.8 BOLTS IN S275

Diameter of Bolt mm	Tensile Stress Area A_t mm ²	Tension Capacity		Shear Capacity		Bearing Capacity in kN (Minimum of P_{bs} and P_{bs})											
		Nominal $0.8A_tP_t$ P_{nom} kN	Exact A_tP_t P_t kN	Single Shear P_s kN	Double Shear $2P_s$ kN	End distance equal to 2 x bolt diameter											
						Thickness in mm of ply passed through.											
						5	6	7	8	9	10	12	15	20	25	30	
12	84.3	37.8	47.2	31.6	63.2	27.6	33.1	38.6	44.2	49.7	55.2	66.2	82.8	110	138	166	
16	157	70.3	87.9	58.9	118	36.8	44.2	51.5	58.9	66.2	73.6	88.3	110	147	184	221	
20	245	110	137	91.9	184	46.0	55.2	64.4	73.6	82.8	92.0	110	138	164	230	276	
22	303	136	170	114	227	50.6	60.7	70.8	81.0	91.1	101	121	152	202	253	304	
24	353	158	198	132	265	55.2	66.2	77.3	88.3	99.4	110	132	166	221	276	331	
27	459	206	257	172	344	62.1	74.5	86.9	99.4	112	124	149	186	248	311	373	
30	561	251	314	210	421	69.0	82.8	96.6	110	124	138	166	207	276	345	414	

Values in **bold** are less than the single shear capacity of the bolt.

Values in *italic* are greater than the double shear capacity of the bolt.

Bearing values assume standard clearance holes.

If oversize or short slotted holes are used, bearing values should be multiplied by 0.7.

If long slotted or kidney shaped holes are used, bearing values should be multiplied by 0.5.

If appropriate, shear capacity must be reduced for large packings, large grip lengths and long joints.

FOR EXPLANATION OF TABLES SEE NOTE 10.1

GRADE 8.8 BOLTS IN S275

Diameter of Bolt mm	Tensile Stress Area A_t mm ²	Tension Capacity		Shear Capacity		Bearing Capacity in kN (Minimum of P_{bs} and P_{bs})											
		Nominal $0.8A_tP_t$ P_{nom} kN	Exact A_tP_t P_t kN	Single Shear P_s kN	Double Shear $2P_s$ kN	End distance equal to 2 x bolt diameter											
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If appropriate, shear capacity must be reduced for large packings, large grip lengths and long joints.

FOR EXPLANATION OF TABLES SEE NOTE 10.1

Base Plate to be used for all columns.