	Project details				
mpa The Concrete Centre	**V1.1 Stocky edge column**	Calculated by	**MM**	Job no.	**V1**
		Checked by	**CIEG**	Sheet no.	**1/12**
		Client	**TCC**	Date	**Nov 2026**

Example

A 325 mm square column on the edge of a flat slab structure supports an axial load of 1620 kN and first order moments of 75 kNm top and −75 kNm bottom about the y-axis, and 25 kNm top and −25 kNm bottom about the z-axis. The concrete is grade C30/37 and cover c_{nom} = 25 mm to stirrups. The 250 mm thick flat slabs are at 4000 mm vertical centres.

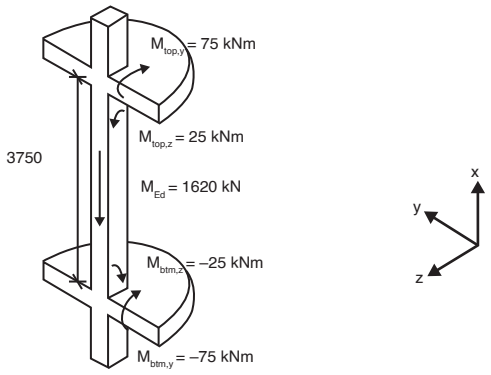


Figure V1.1.1: Actions in edge column

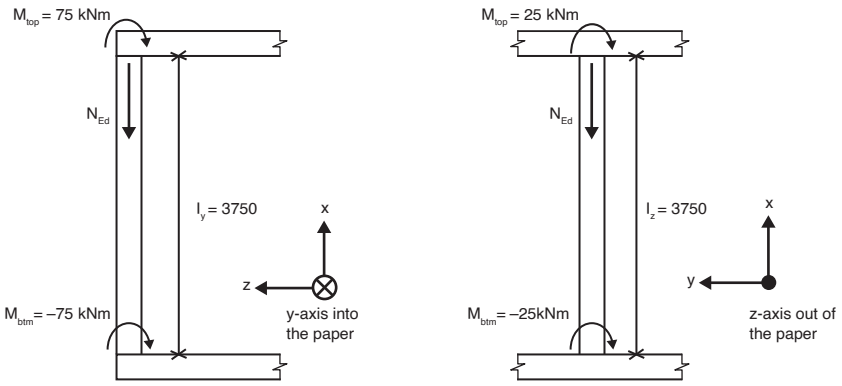


Figure V1.1.2: Actions on the perimeter column in the y- and z-axes

	Project details				
mpa The Concrete Centre	**V1.1 Stocky edge column**	Calculated by	**MM**	Job no.	**V1**
		Checked by	**CIEG**	Sheet no.	**2/12**
		Client	**TCC**	Date	**Nov 2026**

General notes:

The intention of this worked example is to show a typical hand calculation for the design of a stocky column by using design charts, as is typically done in practice. Although it is not a commonly-used method, the same process is repeated using the iteration of the neutral axis distance, x method to determine the reinforcement required.

The calculation takes into account the enhanced concrete compressive strength in the column due to the benefit of confinement.

The load and moment in this worked example are only for one load combination. In practice, all column load combinations, and not only a load envelope, should be considered one by one.

All calculations are carried out in a spreadsheet format where the outcome is not rounded and it is the unrounded value that is used in any further calculation.

This worked example is based on the versions of the codes of practice, guidance documents and publications as noted in the List of References. The user is responsible for ensuring that they use the latest version of the documents.

Abbreviations

General:	Cl.	Clause
	F.	Formula
	Fig.	Figure
	WE	Worked example
Codes of practice:	EC2-1-1	BS EN 1992-1-1:2023 [1]
	EC2-1-2	BS EN 1992-1-2:2023 [2]
	NA EC2-1-1	NA to BS EN 1992-1-1:2026 [3]
	PD 6687	PD 6687:2026 [4]
Guidance documents:	V1. How to	V1 - How to: Stocky columns [5]

mpa The Concrete Centre	Project details **V1.1 Stocky edge column**	Calculated by **MM**	Job no. **V1**
		Checked by **CIEG**	Sheet no. **3/12**
		Client **TCC**	Date **Nov 2026**

V1.1.1 Column effective length

Effective length (for both y- and z-directions):

$l_0 = kl$ — NA EC2-1-1: Cl. NA.4.113

a) About the y-axis

l = clear column length = 4000 – 250/2 – 250/2	=	3750 mm	
k = effective length factor for braced column for condition 3 at top and bottom ends*	=	1.00	NA EC2-1-1: Table NA.9
$\therefore l_{0y} = 3750 \times 1$	=	**3750 mm**	

b) About the z-axis

l = as before	=	3750 mm	
k = effective length factor for braced column for condition 2 at top and bottom ends**	=	0.85	NA EC2-1-1: Table NA.9
$\therefore l_{0z} = 3750 \times 0.85$	=	**3188 mm**	

V1.1.2 Determine first order end moments: M_{01}, M_{02} and M_{0Ed}

> *Note:*
>
> *The first order end moments are denoted M_{01} and M_{02} and include the unfavourable effects of imperfections and the position of loads. $|M_{02}| > |M_{01}|$. M_{0Ed} is the maximum first order moment due to the fundamental load combination, also including the effect of imperfections.* — EC2-1-1: Cl.3.2.1 & NA EC2-1-1: Cl. NA.4.114

$M_{01} = M_{btm} + e_i |N_{Ed}|$ — EC2-1-1: Fig. O.1

$M_{02} = M_{top} + e_i |N_{Ed}|$ — EC2-1-1: Fig. O.1

where:

The effect for imperfections, e_i:

$e_i = \theta_i l_0/2$ — EC2-1-1: F. (7.5)

where:

column inclination, θ_i:

$\theta_i = \alpha_h \alpha_m/200$ — EC2-1-1: F. (7.1)

reduction coefficient for height, α_h:

$0.4 \le \alpha_h = 2/l^{0.5} \le 1.0$ — EC2-1-1: F. (7.2)

For both y- and z-axes:

$l^{\dagger} = 3750/1000$	=	3.75 m	EC2-1-1: Cl. 7.2.1.2 (3)
$2/l^{0.5} = 2/(3.75)^{0.5}$	=	1.0	EC2-1-1: F. (7.2)
$\therefore \alpha_h$	=	1.0	EC2-1-1: F. (7.2)

* Both column ends are connected monolithically to slabs (250 mm thick), which are shallower than the column size (325 mm) in the plane considered. About the y-axis, the column is restrained by the slab on one side only, and hence, condition 2 does not apply. Assuming condition 3 is conservative.

** Both column ends are connected monolithically on either side to slabs (250 mm thick), which are shallower than the column size (325 mm).

† As before (see Section V1.1.1), the length 'l' is the actual column height and is the same for both directions.

mpa The Concrete Centre	Project details **V1.1 Stocky edge column**	Calculated by **MM**	Job no. **V1**
		Checked by **CIEG**	Sheet no. **4/12**
		Client **TCC**	Date **Nov 2026**

reduction coefficient for number of members, α_m:

$\alpha_m = [0.5 \times (1 + 1/m)]^{0.5}$ EC2-1-1: F. (7.3)

$m = 1.0$ EC2-1-1: Cl. 7.2.1.2 (3)

$\therefore \alpha_m = [0.5 \times (1 + 1/1)]^{0.5} = 1.0$

$\therefore \theta_i = 1 \times 1/200 = 0.005$

(Alternatively, use simplification: $\theta_i = 1/200$*) EC2-1-1: Cl. 7.2.1.2 (5b)

$\therefore e_{iy} = e_{iz} = 0.005 \times 3750/2 =$ **9.4 mm**

(Alternatively, use simplification: $e_i = l_0/400$**) EC2-1-1: Cl. 7.2.1.2 (5a)

Summary of first order end moments (including imperfections)

$M_{01y} = -75 + 9.4 \times |1620|/1000 =$ **−59.8 kNm**

$M_{01z} = -25 + 9.4 \times |1620|/1000 =$ **−9.8 kNm**

$M_{02y} = 75 + 9.4 \times |1620|/1000 =$ **90.2 kNm**

$M_{02z} = 25 + 9.4 \times |1620|/1000 =$ **40.2 kNm**

Maximum first order moment including the effect of imperfections, M_{0Ed}:

$M_{0Ed} = C_m M_{02}$ EC2-1-1: F. (O.14)

where:

$C_m = 0.6 + 0.4 r_m \geq 0.4$ EC2-1-1: F. (O.15)

To find r_m, first check if $|M_{02}| < 0.05\ N_{Ed}\ h$† EC2-1-1: F. (O.16)

a) About the y-axis

$0.05\ N_{Ed}\ h = 0.05 \times 1620 \times 325/1000 =$ 26.3 kNm EC2-1-1: F. (O.16)

$|M_{02y}| = 90.2 > 26.3$ EC2-1-1: F. (O.16)

$\therefore r_m = M_{01}/M_{02} = -59.8/90.2 =$ −0.7 EC2-1-1: F. (O.16)

$\therefore C_m = 0.6 + 0.4 \times -0.7 \geq 0.4 =$ 0.40

$\therefore M_{0Edy} = 0.4 \times 90.2 =$ **36.1 kNm**

b) About the z-axis

$|M_{02}| = 40.2 > 26.3$ EC2-1-1: F. (O.16)

$\therefore r_m = M_{01}/M_{02} = -9.8/40.2 =$ −0.2 EC2-1-1: F. (O.16)

$\therefore C_m = 0.6 + 0.4 \times -0.2 \geq 0.4 =$ 0.50

$\therefore M_{0Edz} = 0.5 \times 40.2 =$ **20.2 kNm**

Summary of maximum first order moments (including imperfections)

$M_{0Edy} =$ **36.1 kNm**

$M_{0Edz} =$ **20.2 kNm**

* The simplification corresponds to $\alpha_h = 1.0$ (refer to EC2-1-1 Cl. 7.2.1.2 (5b)).

** In this case, the simplified equations gives an accurate estimation of the effect for imperfections, e_i because $\alpha_h = 1.0$ for a column with $l \leq 4$ m.

† If $M_{02} < 0.05 N_{Ed} h$, then r_m should be taken as 1 and $C = 0.7$ in EC2-1-1 F. (O.12).

mpa The Concrete Centre	Project details **V1.1 Stocky edge column**	Calculated by **MM**	Job no. **V1**
		Checked by **CIEG**	Sheet no. **5/12**
		Client **TCC**	Date **Nov 2026**

V1.1.3 Check slenderness, λ

Slenderness ratio:

$\lambda = l_0/i$ — EC2-1-1: F. (O.8)

where:

i = radius of gyration = $(I_c/A_c)^{0.5} = h/12^{0.5}$ for rectangular sections — EC2-1-1: Cl. O.5 (1)

$\therefore i = 325/12^{0.5}$ = 94 mm*

a) About the y-axis

$\therefore \lambda_y = 3750/94$ = **40.0**

(Alternatively, use How to simplified equation of $\lambda = 3.46 l_0/h$ for rectangular sections) — V1. How to: Fig V1.1

b) About the z-axis

$\therefore \lambda_z = 3188/94$ = **34.0**

(Alternatively, use How to simplified equation of $\lambda = 3.46 l_0/h$ for rectangular sections) — V1. How to: Fig V1.1

V1.1.4 Limiting slenderness, λ_{lim}

Note:

The column is subject to biaxial bending and the slenderness is checked separately in each principal plane of bending.
*For the biaxial bending check, imperfections should be considered in one direction only where they have the most unfavourable effect, according to EC2-1-1 Cl. 7.4.4 (3). However, to determine whether a column is stocky in each direction, imperfections are considered about both the y- and z-axes.***

EC2-1-1: Cl. O.6 (2)

a) About the y-axis

$\lambda_{lim} = 20\ ABC/n^{0.5}$ — EC2-1-1: F. (O.12)

where:

A = (default value) = 0.7 — EC2-1-1: Cl. O.6 (1)

B = (default value) = 1.1 — EC2-1-1: Cl. O.6 (1)

$C = 1.7 - r_m$ — EC2-1-1: Cl. O.6 (1)

where:

r_m (as before from Section V1.1.2) = −0.7

$\therefore C = 1.7 - (-0.7)$ = 2.4

$n = |N_{Ed}|/(A_c f_{cd})$ — EC2-1-1: Cl. O.6 (1)

where f_{cd} can be replaced by the confined concrete strength, $f_{cd,c}$ — EC2-1-1: Cl. 8.1.4

Note:

When concrete is confined, such as in a column, the concrete compressive design strength, f_{cd} may be increased according to EC2-1-1 Cl. 8.1.4. This WE calculates the enhanced confined concrete compressive strength.

The confined concrete compressive strength:

$f_{cd,c} = f_{cd} + k_{conf,b} k_{conf,s} \Delta f_{cd}$ — EC2-1-1: F. (8.15)

where:

f_{cd} = 1 x 1 x 30/1.5 = 20.0 MPa — EC2-1-1: F. (5.3) & (5.4)

$k_{conf,b}$ = effectiveness factor for confinement reinforcement = $1/3(b_{cs}/b)^2$ — EC2-1-1: Table 8.1 & Fig. 8.3 a

* Applicable to both the y- and z-directions because the column section is square and b = h.

** In this WE, should imperfections be excluded from the calculation of M_{02} about the z-axis, a smaller slenderness limit is obtained (refer to†).

† If $M_{02} < 0.05 N_{Ed} h$, then r_m should be taken as 1 and C = 0.7 in EC2-1-1 F. (O.12).

mpa The Concrete Centre	Project details **V1.1 Stocky edge column**	Calculated by **MM**	Job no. **V1**
		Checked by **CIEG**	Sheet no. **6/12**
		Client **TCC**	Date **Nov 2026**

b_{cs} = width of the confinement core
= b – 2c_{nom} = 325 – 2 x 25 = 275 mm
∴ $k_{conf,b}$ = 1/3 x (275 / 325)^2 = 0.24
$k_{conf,s}$ = effectiveness factor for confinement reinforcement = [1 – s/(2b_{cs})]^2 — EC2-1-1: Table 8.1 & Fig. 8.3 a
s = assumed spacing of stirrups = 200 mm
∴ $k_{conf,s}$ = [1 – 200/(2 x 275)]^2 = 0.40

The confinement stress in columns:
$\sigma_{c2d} = (2A_{s,conf}f_{yd})/(b_{cs}s)$ — EC2-1-1: F. (8.11)
where:
$A_{s,conf}$ = cross-sectional ara of one stirrup leg
Assuming 8 mm stirrups:
$A_{s,conf} = \pi \times 8^2/4$ = 50 mm^2
∴ σ_{c2d} = (2 x 50 x 435)/(275 x 200) = 0.79 MPa — EC2-1-1: F. (8.11)
$\sigma_{c2d} < 0.6\ f_{cd}$ = 0.6 x 20 = 12.0 MPa
∴ The compressive strength increase of concrete with $d_{dg} \geq 32$ mm:
$\Delta f_{cd} = 4\sigma_{c2d}$ = 4 x 0.79 = 3.2 MPa — EC2-1-1: F. (8.9)
∴ $f_{cd,c}$ = 20 + 0.24 x 0.4 x 3.2 = **20.3 MPa***
∴ n = 1620 x 1000/(325 x 325 x 20.3) = 0.76
∴ $\lambda_{lim,y}$ = 20 x 0.7 x 1.1 x 2.4/0.76$^{0.5}$ = **41.9**
(Alternatively, use the How to simplified equation: $\lambda_{lim} = 15.4\ C/n^{0.5}$) — V1. How to Fig. V1.1
∴ λ = 40 < 41.9 and the column is stocky
∴ Second order effects may be ignored — EC2-1-1: Cl. O.6 (1)
∴ M_2 = second order moment = **0**

b) About the z-axis

$\lambda_{lim} = 20\ ABC/n^{0.5}$ — EC2-1-1: F. (O.12)
where:
A = (default value) = 0.7 — EC2-1-1: Cl. O.6 (1)
B = (default value) = 1.1 — EC2-1-1: Cl. O.6 (1)
C = 1.7 – r_m — EC2-1-1: Cl. O.6 (1)
where:
r_m (as before from Section V1.1.2) = –0.2
∴ C = 1.7 – (–0.2) = 1.9
n = (as before) = 0.76
∴ $\lambda_{lim,z}$ = 20 x 0.7 x 1.1 x 1.9/0.76$^{0.5}$ = **34.5**
(Alternatively, use the How to simplified equation: $\lambda_{lim} = 15.4\ C/n^{0.5}$) — V1. How to Fig. V1.1
∴ λ = 34 < 34.5 and the column is stocky
∴ Second order effects may be ignored — EC2-1-1: Cl. O.6 (1)
∴ M_2 = second order moment = **0**

V1.1.5 Design moments

$M_{Edy} = \max\{M_{0Ed} + M_2; M_{02}; |M_{01} - 0.5M_2 - 2|N_{Ed}|e_i|\}$ — EC2-1-1: F. (O.13)
where:
M_2 (as before from Section V1.1.4) = 0
∴ $M_{Edy} = \max\{M_{0Ed}; M_{02}; |M_{01} - 2|N_{Ed}|e_i|\}$

* In this WE, the benefit of confinement from the stirrups leads to an increase in the concrete strength of only 1.5%. It is up to the engineer to assess whether it is worth the calculation effort.

mpa The Concrete Centre	Project details **V1.1 Stocky edge column**	Calculated by **MM**	Job no. **V1**
		Checked by **CIEG**	Sheet no. **7/12**
		Client **TCC**	Date **Nov 2026**

a) About the y-axis

As before from Section V1.1.2:

N_{Ed}	=	1620 kN	
M_{01y}	=	–59.8 kNm	
M_{02y}	=	90.2 kNm	
M_{0Edy}	=	36.1 kNm	
e_{iy}	=	9.4 mm	

$\therefore M_{Edy} = \max\{36.1; 90.2; |-59.8 - 2 \times |1620| \times 9.4/1000|\}$ = 90.2 kNm

Check that $M_{Ed} \geq M_{Ed,min}$ — EC2-1-1: Cl. 8.1.1 (5)

$M_{Ed,min} = \pm N_{Ed}\, e_{d,min}$ — EC2-1-1: F. (8.1)

where:

$e_{d,min} = \max[h/30, 20\text{ mm})$

$= \max[325/30, 20\text{ mm}]$ = 20 mm

$\therefore M_{Ed,min} = \pm 1620 \times 20/1000$ = 32.4 kNm

$\therefore$ **M_{Edy} (including imperfections)** = **90.2 kNm**

and M_{Edy} (excluding imperfections) = **75.0 kNm**

b) About the z-axis

As before from Section V1.1.2:

N_{Ed}	=	1620 kN	
M_{01z}	=	–9.8 kNm	
M_{02z}	=	40.2 kNm	
M_{0Edz}	=	20.2 kNm	
e_{iz}	=	9.4 mm	

$\therefore M_{Edz} = \max\{20.2; 40.2; |-9.8 - 2 \times |1620| \times 9.4/1000|\}$ = 40.2 kNm

$> M_{Ed,min}$ = 32.4 kNm

$\therefore$ **M_{Edz} (including imperfections)** = **40.2 kNm**

and M_{Edz} (excluding imperfections) = **30.4 kNm**

V1.1.6 Design using charts

Assuming 8 mm stirrups and 16 mm main bars:

$d = h - c_{nom} - \phi_{links} - \phi/2$

$d = 325 - 25 - 8 - 16/2$ = 284 mm

$d/h = 284/325$ = 0.87

a) About the y-axis

The critical moment = $M = M_{Edy}$ = 90.2 kNm

The corresponding axial force, N = 1620 kN

> *Note:*
>
> *The V1. How to column design charts plot $N_{Ed}/(bhf_{cd,c})$ and $M_{Ed}/(bh^2f_{cd,c})$ where the term f_{cd} may be substituted with $f_{cd,c}$ (from Section V1.1.4)*

$N_{Ed}/(bhf_{cd,c}) = 1620 \times 1000/(325 \times 325 \times 20.3)$ = **0.76**

$M_{Ed}/(bh^2f_{cd,c}) = 90.2 \times 10^6/(325 \times 325^2 \times 20.3)$ = **0.13**

Conservatively using design chart for d/h = 0.85: — V1. How to: Fig. V1.10

$A_s/(bhf_{cd,c})$ = 0.0003 mm²/N

$\therefore A_s = 0.0003 \times 325 \times 325 \times 20.3$ = **643 mm²**

Area of reinforcement per column face = $A_s/2$ = **322 mm²**

For b = 325 mm, minimum number of bars per face for fire = **3** — EC2-1-2: Table D.1

Try 3H12 per column face (339 mm²)

i.e. 8H12 total (905 mm²)

	Project details	Calculated by MM	Job no. V1
mpa The Concrete Centre	**V1.1 Stocky edge column**	Checked by CIEG	Sheet no. 8/12
		Client TCC	Date Nov 2026

Check minimum steel:

$A_{s,min} = \max\{0.1N_{Ed}/f_{yd}, 0.002A_c\}$ EC2-1-1: Table 12.3

where:

$f_{yd} = 500/1.15$ = 435 MPa

$A_{s,min} = \max\{0.1 \times 1620 \times 1000/435, 0.002 \times 325 \times 325\}$ = **373 mm²**

∴ Minimum reinforcement OK

b) About the z-axis

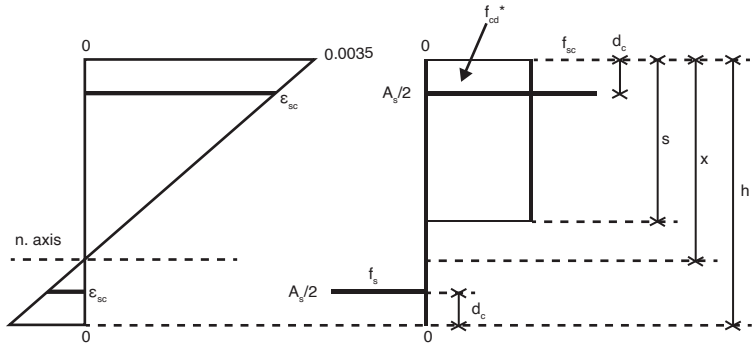
By inspection 3H12 per column face is OK

Try 8H12 total (905 mm²)

V1.1.7 Design using iteration of x

Note:

An alternative method to using charts in Section V1.1.6 is to use iteration of the neutral axis, x.



a) Strain diagram b) Stress diagram

Figure V1.1.3: Section in axial compression and bending

Assume that $d_c = h - d$

Consider the equivalent stress block in Figure V1.1.3**

By considering equilibrium in the steel and concrete forces and rearranging:

$A_{s,N} = 2(N - f_{cd}\, bs)/(f_{sc} - f_s)$

By considering moment equilibrium in the steel and concrete and rearranging:

$A_{s,M} = 2[(M - f_{cd}bs)(h/2 - s/2)]/[(f_{sc} + f_s)(d - h/2)]$

where:

The critical moment = M = 90.2 kNm

The corresponding axial force, N = 1620 kN

$d_c = 25 + 8 + 16/2$ = 41 mm

$d = 325 - 41$ = 284 mm

* As before, f_{cd} may be replaced with $f_{cd,c}$

** For simplicity consider only reinforcement at the most compressed and tensile sides.

mpa The Concrete Centre	Project details **V1.1 Stocky edge column**	Calculated by **MM**	Job no. **V1**
		Checked by **CIEG**	Sheet no. **9/12**
		Client **TCC**	Date **Nov 2026**

Start with a trial value for the depth of neutral axis, say x	=	280 mm	
s = 0.8 x 280 but ≤ h	=	224 mm	
ε_{sc} = 0.0035(x – d_c)/x but ≤ f_{yd}/E_s			
= 0.0035(280 – 41)/280	=	0.002988	
f_{yd}/E_s = 435/200000	=	0.002174	
∴ ε_{sc} = min(0.002988, 0.002174)	=	0.002174	
f_{sc} = 200000 x 0.002174	=	435 MPa	
ε_s = 0.0035(d – x)/ x but ≤ f_{yd}/E_s			
= 0.0035(284 – 280)/280 but ≤ 0.002174	=	0.000050	
f_s = 200000 x 0.00005	=	10 MPa	
∴ $A_{s,N}$ = 2 x (1620 x 10^3 – 20.3 x 325 x 224)/(435 – 10)	=	667 mm²	
∴ $A_{s,M}$ = 2 x [90.2 x 10^6 – (20.3 x 325 x 224) x (325/2 – 224/2)]/ [(435 + 10)(284 – 325/2)]	=	575 mm²	
Refine x until $A_{s,M} \approx A_{s,N}$			
x	=	282.1 mm	
s = 0.8 x 282.1 but ≤ h	=	225.68 mm	
ε_{sc} = 0.0035(282.1 – 41)/282.1 but ≤ 0.002174	=	0.002174	
f_{sc} = 200000 x 0.002174	=	435 MPa	
ε_s = 0.0035(d – x)/x but ≤ f_{yd}/E_s			
= 0.0035(284 – 282.1)/282.1 but ≤ 0.002174	=	0.000024	
f_s = 200000 x 0.000024	=	5 MPa	
$A_{s,N}$ = 2 x (1620 x 10^3 – 20.3 x 325 x 225.7)/(435 – 5)	=	607 mm²	
$A_{s,M}$ = 2 x [90.2 x 10^6 – (20.3 x 325 x 225.7) x (325/2 – 225.7/2)]/ [(435 + 5)(284 – 325/2)]	=	608 mm²	
∴ $A_{s,M} \approx A_{s,N}$			
∴ x ≈ 282.1 mm			
∴ Provide A_s	=	**608 mm²**	
Area of reinforcement per column face = $A_s/2$	=	**304 mm²**	

Try 3H12 per column face (339 mm²)
i.e. 8H12 total (905 mm²)

Note:
The reinforcement area obtained from the iteration method and that obtained from design charts is similar.

V1.1.8 Check for biaxial bending

Note:
Biaxial bending checks may be omitted if F. (7.29) and F. (7.30) are satisfied, i.e.:
$0.5 \le \lambda_y / \lambda_z \le 2.0$ and
$e'_y / e'_z \le 0.2$ or $e'_y / e'_z \ge 5.0$

EC2-1-1: Cl. 7.4.4 (4)
EC2-1-1: F. (7.29)
EC2-1-1: F. (7.30)

λ_y/λ_z = 40/34	=	1.2
∴ **Formula (7.29) is satisfied**		

mpa The Concrete Centre	Project details **V1.1 Stocky edge column**	Calculated by **MM**	Job no. **V1**
		Checked by **CIEG**	Sheet no. **10/12**
		Client **TCC**	Date **Nov 2026**

Consider imperfections in the y-axis only:

The dimensionless eccentricities about both axes are: EC2-1-1: Cl. 7.4.4 (4) & Fig. 7.5

$e'_z = M_{Edy}/|N_{Ed}b| = 90.2/|1620 \times 0.325| = 0.17$

$e'_y = M_{Edz}/|N_{Ed}h| = 30.4/|1620 \times 0.325| = 0.06$

$\therefore e'_y/e'_z = 0.06/0.17 = 0.34$

$\therefore$ **Formula (7.30) is not satisfied**

$\therefore$ Biaxial bending should be taken into account EC2-1-1: Cl. 7.4.4 (5)

> *Note:*
>
> *EC2-1-1 Cl. 7.4.4 (3) states that for the biaxial bending check, imperfections should be considered in one direction only where they have the most unfavourable effect.*

Check if the following simplified criterion is satisfied: EC2-1-1: Cl. 8.1.1 (8)

$(|M_{Edz}|/M_{Rdz,N})^{a_N} + (|M_{Edy}|/M_{Rdy,N})^{a_N} \leq 1.0$ EC2-1-1: F. (8.2)

where:

a_N is dependent on the N_{Ed} and the axial concrete capacity, $N_{Rd,0}$ EC2-1-1: Cl. 8.1.1 (8)

$N_{Rd,0} = A_c f_{cd} + A_s f_{yd}$ EC2-1-1: F. (8.3)

where the concrete strength, f_{cd} can be replaced with $f_{cd,c}$ EC2-1-1: Cl. 8.1.1 (8)

$\therefore N_{Rd,0} = A_c f_{cd,c} + A_s f_{yd}$ EC2-1-1: F. (8.3)

$\therefore N_{Rd,0} = [(325 \times 325 \times 20.3) + (905 \times 435)]/1000 = 2538$ kN*

$|N_{Ed}|/N_{Rd,0} = 1620/2538 = 0.6$

$\therefore a_N$ (by interpolation) $= 1.45$ EC2-1-1: Cl. 8.1.1 (8)

$N/(bhf_{cd,c}) =$ (as before from Section V1.1.6) $= 0.76$

$A_s/(bhf_{cd,c}) = 679/(325 \times 325 \times 20.3) = 0.00032$ mm^2/N

Conservatively using design charts for d/h = 0.85, for $A_s/(bhf_{cd}) = 0.03\%$: V1. How to: Fig. V1.10

$M_{Rd}/(bh^2 f_{cd,c}) = 0.13$

$\therefore M_{Rdz,N} = M_{Rdy,N} = 0.13 \times 325 \times 325^2 \times 20.3/10^6 = 91$ kNm

Consider imperfections in the y-axis only: EC2-1-1: Cl. 7.4.4 (3)

$\therefore (|M_{Edz}|/M_{Rdz,N})^{a_N} + (|M_{Edy}|/M_{Rdy,N})^{a_N}$

$= (|30.4|/91)^{1.45} + (|90.2|/91)^{1.45} =$ **1.20**

> 1.0

Consider imperfections in the z-axis only: EC2-1-1: Cl. 7.4.4 (3)

$\therefore (|M_{Edz}|/M_{Rdz,N})^{a_N} + (|M_{Edy}|/M_{Rdy,N})^{a_N}$

$= (|40.2|/91)^{1.45} + (|75|/91)^{1.45} =$ **1.07**

> 1.0

$\therefore$ Increase reinforcement area to 3H16 per face (603 mm^2)
i.e. 8H16 total (1609 mm^2)

* $N_{Rd,0}$ is the design value of axial resistance under compression without any accompanying moments and thus, A_s in F. (8.3) is the total reinforcement area of all 8 bars. This is unlike the value of A_s considered in the N-M charts, where only bars at opposite faces are considered (i.e. 6 bars).

mpa The Concrete Centre	Project details **V1.1 Stocky edge column**	Calculated by **MM**	Job no. **V1**
		Checked by **CIEG**	Sheet no. **11/12**
		Client **TCC**	Date **Nov 2026**

With the increased reinforcement:

$N_{Rd,0}$ = (325 x 325 x 20.3) + 1608 x 435/1000 = 2844 kN

$|N_{Ed}|/N_{Rd,0}$ = 1620/2844 = 0.6

$\therefore a_N$ (by interpolation) = 1.39 EC2-1-1: Cl. 8.1.1 (8)

$N/(bhf_{cd,c})$ = (as before from Section V1.1.6) = 0.76

$A_s/(bhf_{cd,c})$ = 1206/(325 x 325 x 20.3) = 0.00056 mm^2/N

Conservatively using design charts for d/h = 0.85, for A_s/bhf_{cd} = 0.05%:

$M_{Rd}/(bh^2f_{cd,c})$ = 0.155

$\therefore M_{Rdz,N} = M_{Rdy,N}$ = 0.155 x 325 x 325^2 x/10^6 = 108 kNm

Consider imperfections in the y-axis only: EC2-1-1: Cl. 7.4.4 (3)

$\therefore (|M_{Edz}|/M_{Rdz,N})^{a_N} + (|M_{Edy}|/M_{Rdy,N})^{a_N}$

= $(|30.4|/108)^{1.39} + (|90.2|/108)^{1.39}$ = **0.95**

< 1.0

Consider imperfections in the z-axis only: EC2-1-1: Cl. 7.4.4 (3)

$\therefore (|M_{Edz}|/M_{Rdz,N})^{a_N} + (|M_{Edy}|/M_{Rdy,N})^{a_N}$

= $(|40.2|/108)^{1.39} + (|75|/108)^{1.39}$ = **0.85**

< 1.0

$\therefore$ Formula (8.2) is satisfied and no further checks are required

$\therefore$ Biaxial check OK for 8H16

V1.1.9 Links

Minimum link bar diameter = 0.25ϕ = 0.25 x 16 = 4.0 mm EC2-1-1: Table 12.3

$\therefore$ Use 8 mm diameter stirrups

At intermediate region, maximum spacing = $s_{max,col}$ = 20$\phi \leq$ min{h, b, 300 mm} EC2-1-1: Table 12.3

$\therefore s_{max,col}$ = 20 x 16 $\leq$ min{325, 325, 300} = 300 mm

At the end regions, maximum spacing = 0.6 $s_{max,col}$ EC2-1-1: Table 12.3

= 0.6 x 300 = 180 mm

over a length of 325 mm

For simplicity, use a spacing of 175 mm over the entire column height

Number of legs:

Bars are at 121.5 mm cc i.e. <150 mm. Therefore, no need to restrain bars in face EC2-1-1: Cl. 12.6 (3)

Note:

The assumptions for the main reinforcement and the links diameters are as assumed and thus, no further checks are required.

	Project details	Calculated by MM	Job no. V1
mpa The Concrete Centre	**V1.1 Stocky edge column**	Checked by CIEG	Sheet no. 12/12
		Client TCC	Date Nov 2026

V1.1.10 Design summary

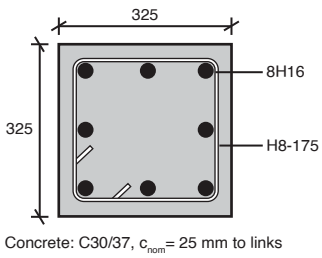


Figure V1.1.4: Design summary for edge column

List of references

[1] British Standards Institution. (2023) BS EN 1992-1-1:2023 Eurocode 2 – Design of concrete structures. Part 1-1: General rules and rules for buildings, bridges and civil engineering structures. London, BSI.

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[4] British Standards Institution. (2026) PD 6687:2026 Background paper to the National Annexes to BS EN 1992-1-1:2023, BS EN 1992-1-2:2023 and BS EN 1992-4:2018. London, BSI.

[5] MPA The Concrete Centre. (2026) How to design concrete structures using Eurocode 2 (second generation). V1 - How to: Stocky Columns. London, TCC.

Revision number	Revision date	Notes
1.0	Nov 2026	First publication

At the time of publication, work on the second generation of Eurocodes and supporting documents is ongoing. Where possible, this worked example has been developed based on the standards that have been published to date, but in some instances, the relevant documents have not yet been finalised and so anticipated values are included and clearly noted as such. Amendments are planned for all of the Eurocodes ahead of the March 2028 date of adoption and users are advised to ensure they are referring to the latest version of a standard or publication.

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