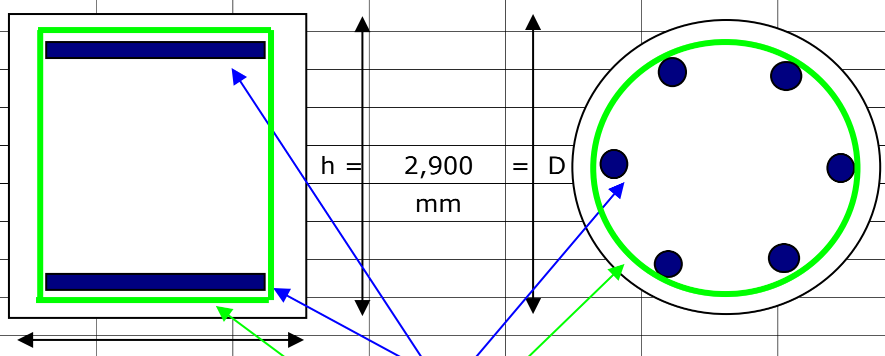
CONSULTING ENGINEERS	Engineering Calculation Sheet Consulting Engineers	Job No.	Sheet No.	Rev.
		jXXX	1	
		Member/Location		
Job Title	Member Design - Reinforced Concrete Column Slendern	Drg. Ref.		
Member Design - RC Column Slenderness Effects		Made by XX	Date 26/8/2024	Chd.

Material Properties

Characteristic strength of concrete (column), f_{cu} | f_{ck}/f_c' (f_{cu} ≤ 105N	80	65	N/mm^2	OK
Yield strength of longitudinal steel, f_y	Higher	500	N/mm^2	
Yield strength of shear link steel, f_{yv}	Higher	500	N/mm^2	
Modulus of elasticity, $E_c = 20+0.2f_{cu}$ | $22[(f_{ck}+8)/10]^{0.3}$ | $4700\sqrt{f_c'}$	ACI318	37.9	kN/mm^2	*9.2.2.1 ACI*

Section Dimensions

Section type		Rectangular	
Depth (larger), h (rectangular) or diameter, D (circular)		2,900	mm
Width (smaller), b (rectangular) or N/A (circular)		900	mm
Area of section, A_c = b.h (rectangular) or $\pi D^2/4$ (circular)		2.610	m^2
	Major	**Minor**	
Second moment of area, $I_{x\|y} = b.h^3/12$ | $h.b^3/12$ | $\pi D^4/64$	1.829	0.176	m^4
Radius of gyration, $r_{x\|y} = \sqrt{(I_{x\|y}/A_c)}$	837	260	mm
Location internal (=2.0) or edge (=1.0), IE — Internal | Internal	2	2	



h = 2,900 mm = D			
b = 900 mm	A_{sc} =	62H32	
	Links	Rebars =	500 MPa
Cover =	25 mm	Links =	500 MPa
Concrete =	C65/80	Steel % =	1.91 %

Effective Depth and Width

	Major	**Minor**	
No.s of layers of steel at each extremity for rect cols, $n_{layers,h}$ | n_{laye}	6	2	layer(s)
Spacer reinforcement, s_r = MAX (ϕ, 25mm, user)	85	85	mm
Effective depth, h' = h - $cover_{main}$ - [ϕ+($n_{layers,h}$-1)(ϕ+s_r)]/2 rect	88%	2,555	mm
= D - $cover_{main}$ - ϕ/2 circular			
Effective width, b' = b - $cover_{main}$ - [ϕ+($n_{layers,b}$-1)(ϕ+s_r)]/2 rect	88%	789	mm
= D - $cover_{main}$ - ϕ/2 circular			

Longitudinal and Shear Reinforcement Details

Longitudinal steel reinforcement diameter, ϕ	32	mm
Longitudinal steel area % (uniaxial), $[A_{sc}/A_c]$.100%	1.91	
Longitudinal steel area % (orthogonal), $[A_{sc+}/A_c]$.100%	0.00	%
Total longitudinal steel area % (uniaxial+orthogonal), $\{[A_{sc}+A_{sc+}]/A_c\}$.100%	1.91	%
Total longitudinal steel area provided, $A_{sc}+A_{sc+} = \{[A_{sc}+A_{sc+}]/A_c\}.100\%.A_c$	49,863	mm^2
Total longitudinal steel reinforcement number, $n_l = [A_{sc}+A_{sc+}]/(\pi.\phi^2/4)$	62	H32
Shear link diameter, ϕ_{link}	12	mm
Cover to all reinforcement, cover (usually 35 (C35) or 30 (C40) internal; 40 e	25	mm
Cover to main reinforcement, $cover_{main}$ = cover + ϕ_{link}	37	mm

CONSULTING ENGINEERS	Engineering Calculation Sheet Consulting Engineers	Job No.	Sheet No.	Rev.
		jXXX	2	
		Member/Location		
Job Title	Member Design - Reinforced Concrete Column Slendern	Drg. Ref.		
Member Design - RC Column Slenderness Effects		Made by XX	Date 26/8/2024	Chd.

Effects From Structural Analysis	Do not adopt code equivalence			**Note**
Axial force, N (tension -ve and comp +ve) (ensure >= 0)		100,000	kN	**OK**
	Major	**Minor**		
Shear force, V_y | V_z	0	0	kN	**Note**
Primary bending moment at top, $M_{xp,t}$ | $M_{yp,t}$	5,000	1,000	kNm	**Note**
Primary bending moment at bot., $M_{xp,b}$ | $M_{yp,b}$	3,000	500	kNm	**Note**
Primary bending moment min., M_1=MIN{$M_{xp,t}$, $M_{xp,b}$}|MIN{$M_{yp,t}$, M	3,000	500	kNm	
Primary bending moment max., M_2=MAX{$M_{xp,t}$, $M_{xp,b}$}|MAX{$M_{yp,t}$, M	5,000	1,000	kNm	
Restraint Dimensions				
	Major	**Minor**		
Column storey (floor to floor) height, l_{storey}	4,500	4,500	mm	
Column clear height, $l_{clear,x\|y} = l_{storey} - h_{rest}$	**4,050**	**4,050**	mm	
Restraint depth, h_{rest}	450	450	mm	
Restraint width, b_{rest}	3,000	3,000	mm	
Restraint span, l_{rest}	8,500	8,500	mm	
Restraint sec. moment of area, $I_{rest} = b_{rest}.h_{rest}^3/12$	0.023	0.023	m^4	
Stiffness parameter, $k_{1,x\|y}$=$k_{2,x\|y}$=($I_{x\|y}$/$l_{clear,x\|y}$)/[IE.Σ2(I_{rest}/l_{rest})]	4.2E+01	4.1E+00		*MOSLEY*

Effective Depth and Width Calculation With Multiple Layers of Steel at Each Extremity

Note that the no. of layer, n_{layers} may be increased to include layers beyond the face of the column and up to and until the centroid of the section, user-spaced accordingly, with the lumping of all rebar within the uniaxial bending steel area, A_{sc} without any orthogonal bending steel area, A_{sc+}, such that the ratio of h'/h or b'/b ≈ 75%, whichever relevant;

Where reinforcement is not concentrated in the corners, a conservative approach is to calculate an effective value of d_2 as illustrated in Figure 15.4.
d_2 = effective depth to steel in layer 2

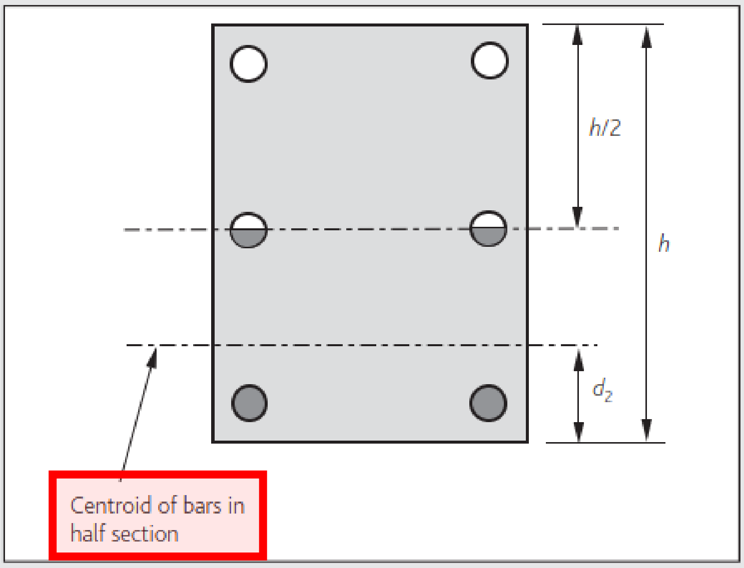


Figure 15.4
Method of assessing d_2 including side bars

CONSULTING ENGINEERS	Engineering Calculation Sheet Consulting Engineers	Job No.	Sheet No.	Rev.
		jXXX	3	
		Member/Location		
Job Title	Member Design - Reinforced Concrete Column Slendern	Drg. Ref.		
Member Design - RC Column Slenderness Effects		Made by XX	Date 26/8/2024	Chd.

Summary of Effects From Slenderness Analysis

		Major	Minor		
Braced or unbraced column ?		Braced	Braced		

Note braced or unbraced column affects slenderness limits criteria, effective height factor and slenderness mag

Note braced = {column / wall stabilized by other bracing, shear walls or core walls and outriggers};

Note unbraced = {column / wall stabilized by bending in itself (columns in moment frames, tube major plane

Note unbraced cantilever = {column / wall stabilized by bending in itself (shear wall major plane, cantilever p

Axial force, N (tension -ve and comp +ve) (ensure >= 0)		100,000	kN	
Pure axial cap., $N_0 = 0.45f_{cu}.A_c+0.95f_y.(A_{sc}+A_{sc+})$	BS110	115,750	kN	86%
Code axial cap., $N_{cap} = 0.40f_{cu}.A_c+0.80f_y.(A_{sc}+A_{sc+})$	BS110	101,870	kN	98%
Code axial stress wrt strength ratio, $[N_{cap}/A_c]/f_{cu}$		49%		
Axial force, N (tension -ve and comp +ve) (ensure >= 0)		100,000	kN	
Pure axial cap., $N_0 = \eta.0.567f_{ck}.A_c+0.87f_y.(A_{sc}+A_{sc+})$	EC2	110,668	kN	90%
Code axial cap., $N_{cap} = \eta.0.50f_{ck}.A_c+0.76f_y.(A_{sc}+A_{sc+})$	EC2	97,411	kN	103%
Code axial stress wrt strength ratio, $[N_{cap}/A_c]/f_{cu}$		47%		
Axial force, N (tension -ve and comp +ve) (ensure >= 0)		100,000	kN	
Pure axial cap., $N_0 = 0.80.[0.85f_c'.[A_c-(A_{sc}+A_{sc+})]+f_y.(A_{sc}+A_{sc+})]$	ACI318	133,103	kN	75%
Code axial cap., $N_{cap} = \phi 0.80.[0.85f_c'.[A_c-(A_{sc}+A_{sc+})]+f_y.(A_{sc}+A_{sc+})]$	ACI318	86,517	kN	116%
Code axial stress wrt strength ratio, $[N_{cap}/A_c]/f_{cu}$		41%		

		Major	Minor		
Design moment, $M_{x\|y} = MAX\{\|M_{xp\|yp}\|+M_{add,x\|y}, M_{e,x\|y}\}$	BS110	5,000	2,000	kNm	
Design moment, $M_{biaxial} = M_x+\beta.(h'/b').M_y$ or $M_y+\beta.(b'/h$	BS110	N/A	2,684	kNm	*cl.3.8.4.5*
Design moment, $M_{x\|y} = MAX\{\|M_{xp\|yp}\|+N.e_{a,x\|y}+M_{add,x\|y},$	EC2	9,667	3,000	kNm	
Design moment, $M_{biaxial} = M_x+\beta.(h'/b').M_y$ or $M_y+\beta.(b'/h$	EC2	N/A	4,225	kNm	*9.3 MOSLE*
Design moment, $M_{x\|y} = MAX\{\|M_{xp\|yp}\|+M_{add,x\|y}, M_{e,x\|y}\}$	ACI318	10,200	4,200	kNm	
Design moment, $M_{biaxial} = M_x+\beta.(h'/b').M_y$ or $M_y+\beta.(b'/h$	ACI318	N/A	5,145	kNm	*9.3 MOSLE*

Design axial stress, σ_N = N/bh | N/D^2	BS110	38.3	N/mm^2	
Design axial stress ratio, σ_N/f_{cu}	BS110	0.48		
Enh. coeff. for biaxial bending, β	BS110	0.44		*cl.3.8.4.5*
Design axial stress, σ_N = N/bh | N/D^2	EC2	38.3	N/mm^2	
Design axial stress ratio, σ_N/f_{ck}	EC2	0.59		
Enh. coeff. for biaxial bending, $\beta=1-\sigma_N/f_{ck}$	EC2	0.41		*T.9.3 MOSLE*
Design axial stress, σ_N = N/bh | N/D^2	ACI318	38.3	N/mm^2	
Design nominal axial stress ratio, $\sigma_N/[\phi.f_c']$	ACI318	0.91		
Enh. coeff. for biaxial bending, $\beta=1-\sigma_N/[\phi.f_c']$	ACI318	0.30		*T.9.3 MOSLE*

Uniaxial or biaxial bending ?		Uniaxial		
Design 1/6th bending stress, $\sigma_{Mx\|y} = M_x/bh^2$ | M_y/hb^2 | M	BS110	0.66	0.85	N/mm^2
Design 1/6th bending stress ratio, $\sigma_{Mx\|y}/f_{cu}$	BS110	0.01	0.01	
Design 1/6th bending stress, $\sigma_{Mx\|y} = M_x/bh^2$ | M_y/hb^2 | M	EC2	1.28	1.28	N/mm^2
Design 1/6th bending stress ratio, $\sigma_{Mx\|y}/f_{ck}$	EC2	0.02	0.02	
Design 1/6th bending stress, $\sigma_{Mx\|y} = M_x/bh^2$ | M_y/hb^2 | M	ACI318	1.35	1.79	N/mm^2
Design 1/6th nominal bending stress ratio, $\sigma_{Mx\|y}/[\phi.f_c']$	ACI318	0.03	0.04	
Effective depth to dimension ratio, h'/(h|D) | b'/(b|D)		*0.88*	*0.88*	

		Major	Minor		
Longitudinal steel area ratio, $[A_{sc}/A_c].f_y/f_{cu}$	BS110	0.08	0.08		*Int. Chart*
Longitudinal steel area %, $[A_{sc}/A_c]$.100%={$[A_{sc}/A_c].f_y/f$	BS110	1.39	1.39	%	OK
Longitudinal steel area ratio, $[A_{sc}/A_c].f_y/f_{ck}$	EC2	0.08	0.08		*Int. Chart*
Longitudinal steel area %, $[A_{sc}/A_c]$.100%={$[A_{sc}/A_c].f_y/f$	EC2	1.04	1.04	%	OK
Longitudinal steel area ratio, $[A_{sc}/A_c]$	ACI318	0.016	0.021		*Int. Chart*
Longitudinal steel area %, $[A_{sc}/A_c]$.100%	ACI318	1.60	2.10	%	NOT OK

CONSULTING ENGINEERS	Engineering Calculation Sheet Consulting Engineers	Job No. jXXX	Sheet No. 4	Rev.
		Member/Location		
Job Title	Member Design - Reinforced Concrete Column Slendern	Drg. Ref.		
Member Design - RC Column Slenderness Effects		Made by **XX**	Date **26/8/2024**	Chd.

BS8110

Interaction Chart [Major Plane] **Major**

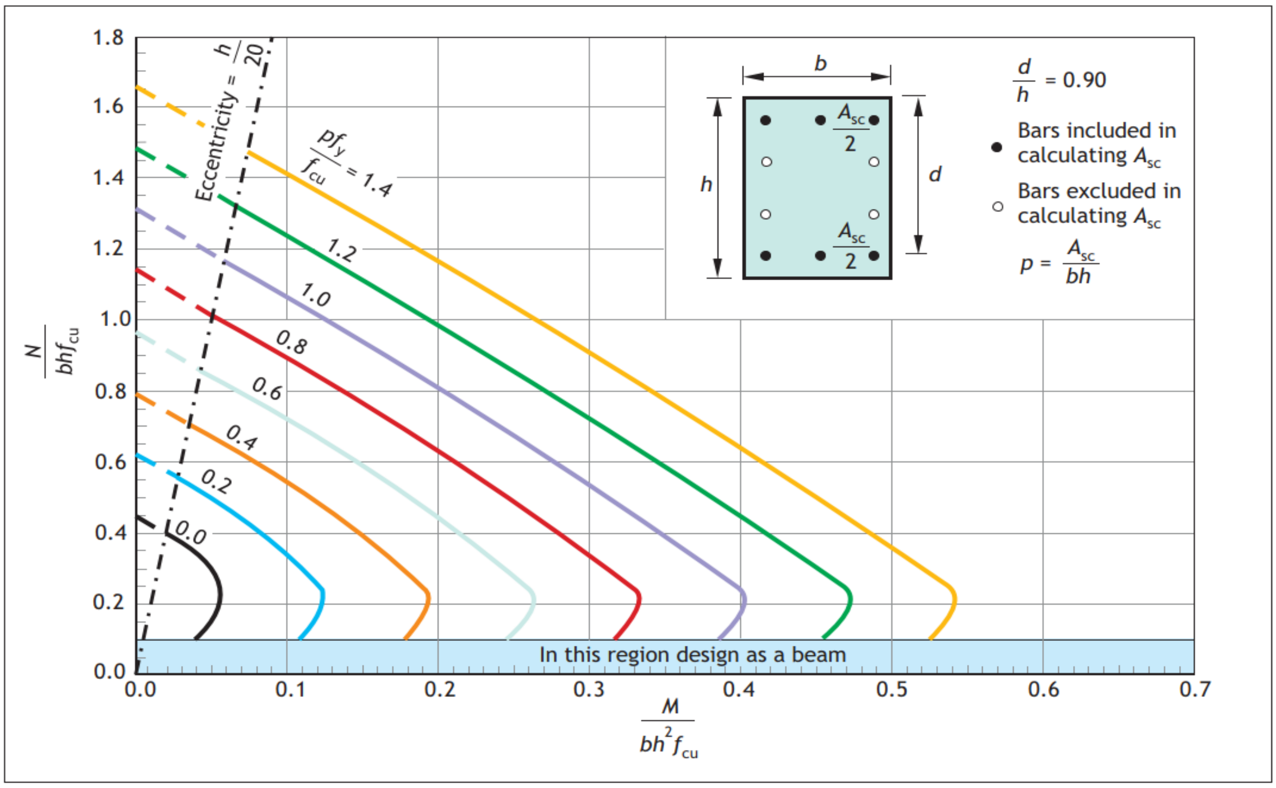


Figure C.4b
Column design chart for rectangular column *d/h* = 0.90

Design axial stress ratio, σ_N/f_{cu}	**BS110**	**0.48**
Design 1/6th bending stress ratio, σ_{Mx}/f_{cu}		**0.01**

Y

Interaction Chart [Minor Plane] **Minor**

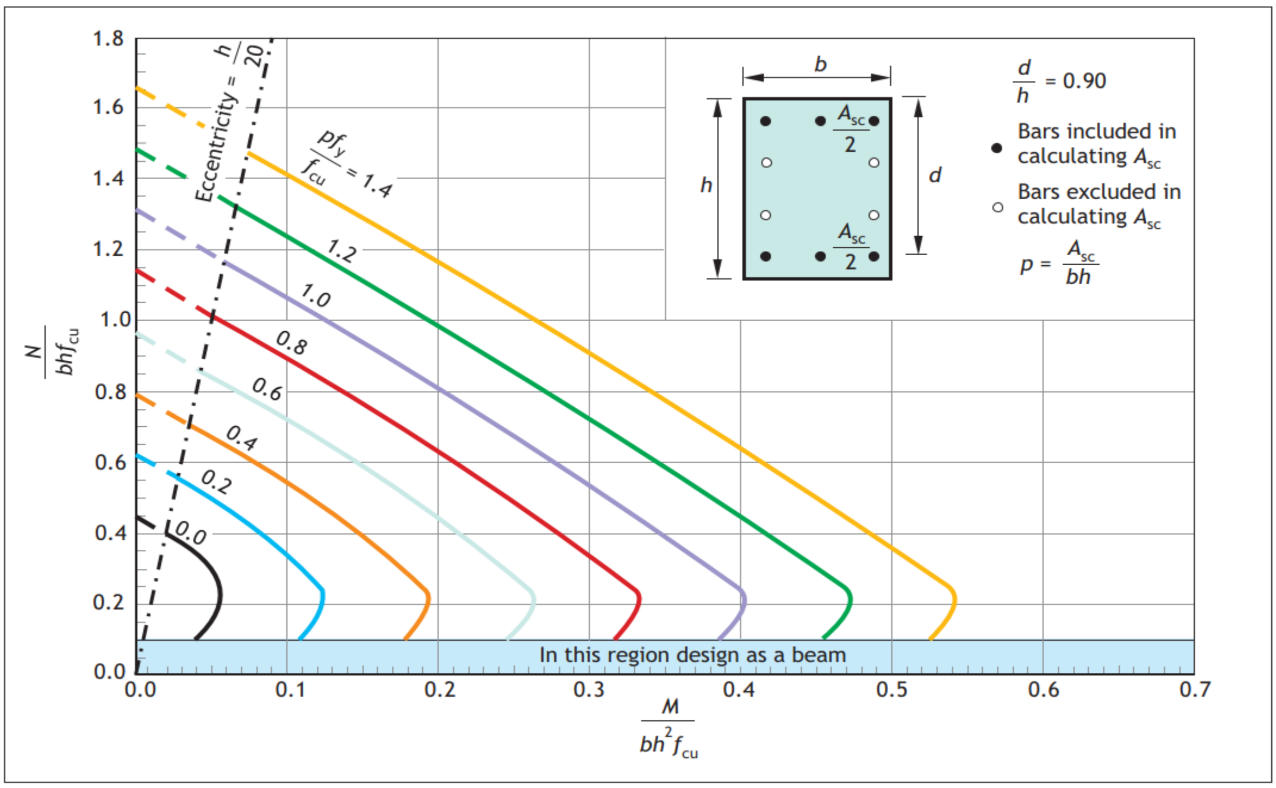


Figure C.4b
Column design chart for rectangular column *d/h* = 0.90

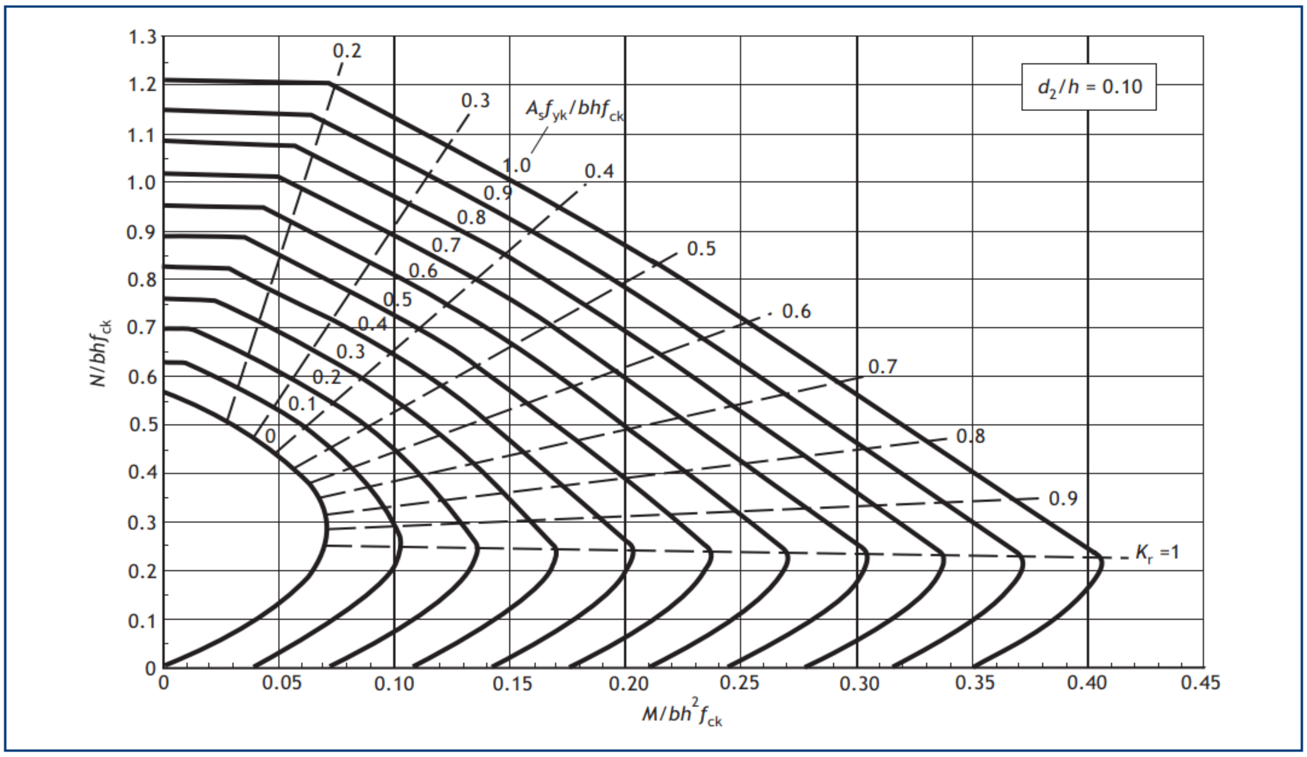
Design axial stress ratio, σ_N/f_{cu}	**BS110**	**0.48**
Design 1/6th bending stress ratio, σ_{My}/f_{cu}		**0.01**

CONSULTING ENGINEERS	Engineering Calculation Sheet Consulting Engineers	Job No.	Sheet No.	Rev.
		jXXX	5	
		Member/Location		
Job Title	Member Design - Reinforced Concrete Column Slendern	Drg. Ref.		
Member Design - RC Column Slenderness Effects		Made by XX	Date 26/8/2024	Chd.

EC2

Interaction Chart [Major Plane] Major

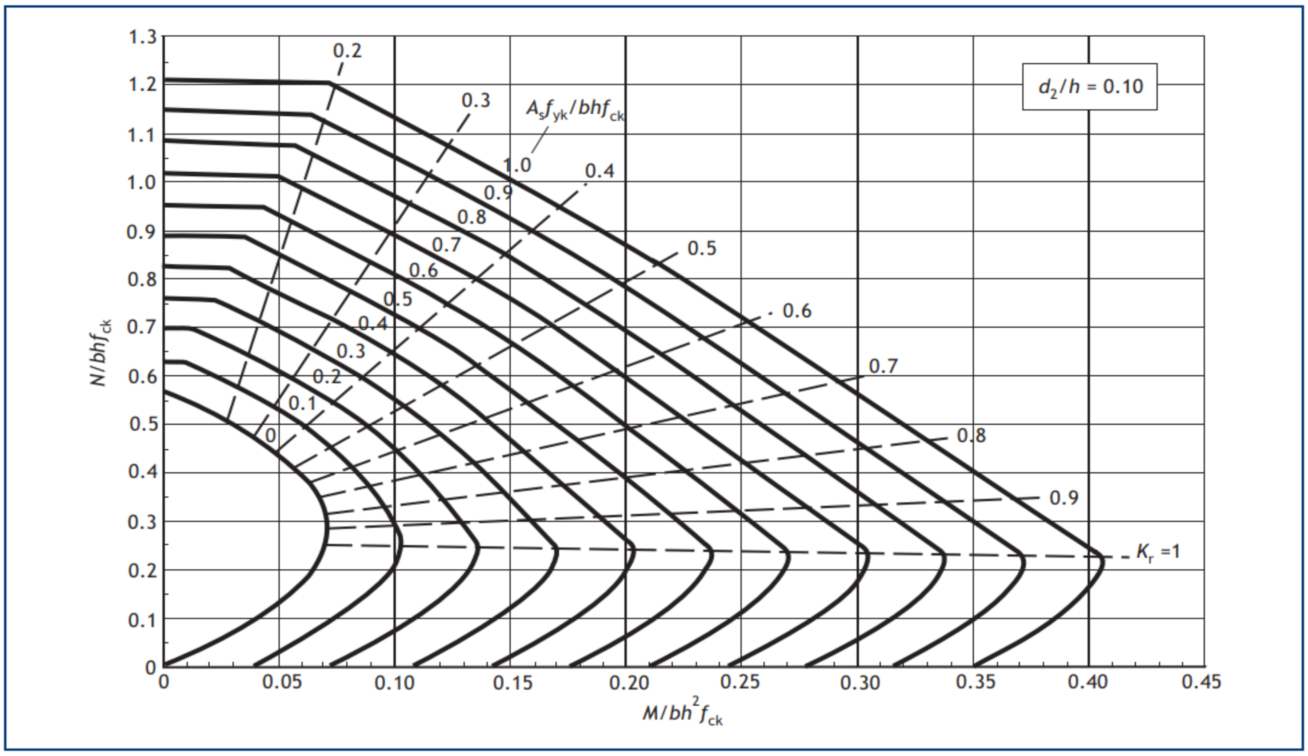
Figure 9b
Column design chart for rectangular columns $d_2/h = 0.10$



Design axial stress ratio, σ_N/f_{ck}	EC2	0.59
Design 1/6th bending stress ratio, σ_{Mx}/f_{ck}		0.02

Interaction Chart [Minor Plane] Minor

Figure 9b
Column design chart for rectangular columns $d_2/h = 0.10$

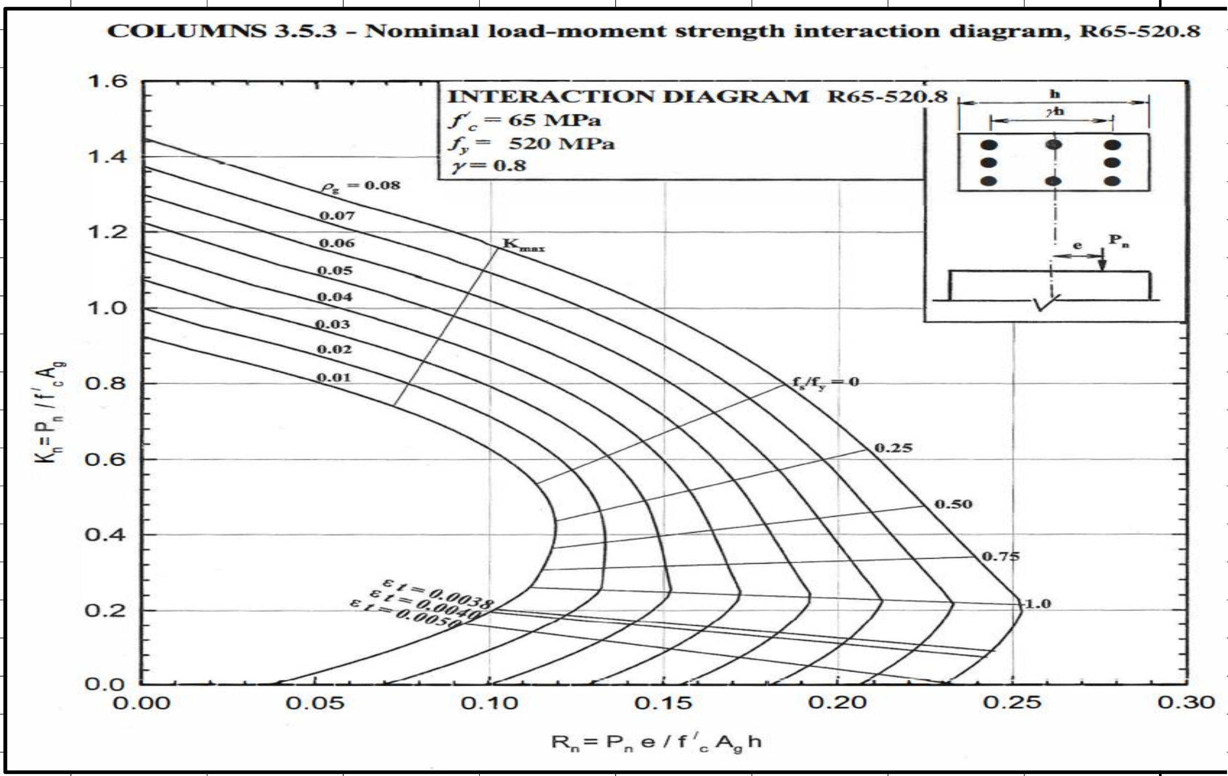


Design axial stress ratio, σ_N/f_{ck}	EC2	0.59
Design 1/6th bending stress ratio, σ_{My}/f_{ck}		0.02

CONSULTING ENGINEERS	Engineering Calculation Sheet Consulting Engineers	Job No.	Sheet No.	Rev.
		jXXX	6	
		Member/Location		
Job Title	Member Design - Reinforced Concrete Column Slendern	Drg. Ref.		
Member Design - RC Column Slenderness Effects		Made by XX	Date 26/8/2024	Chd.

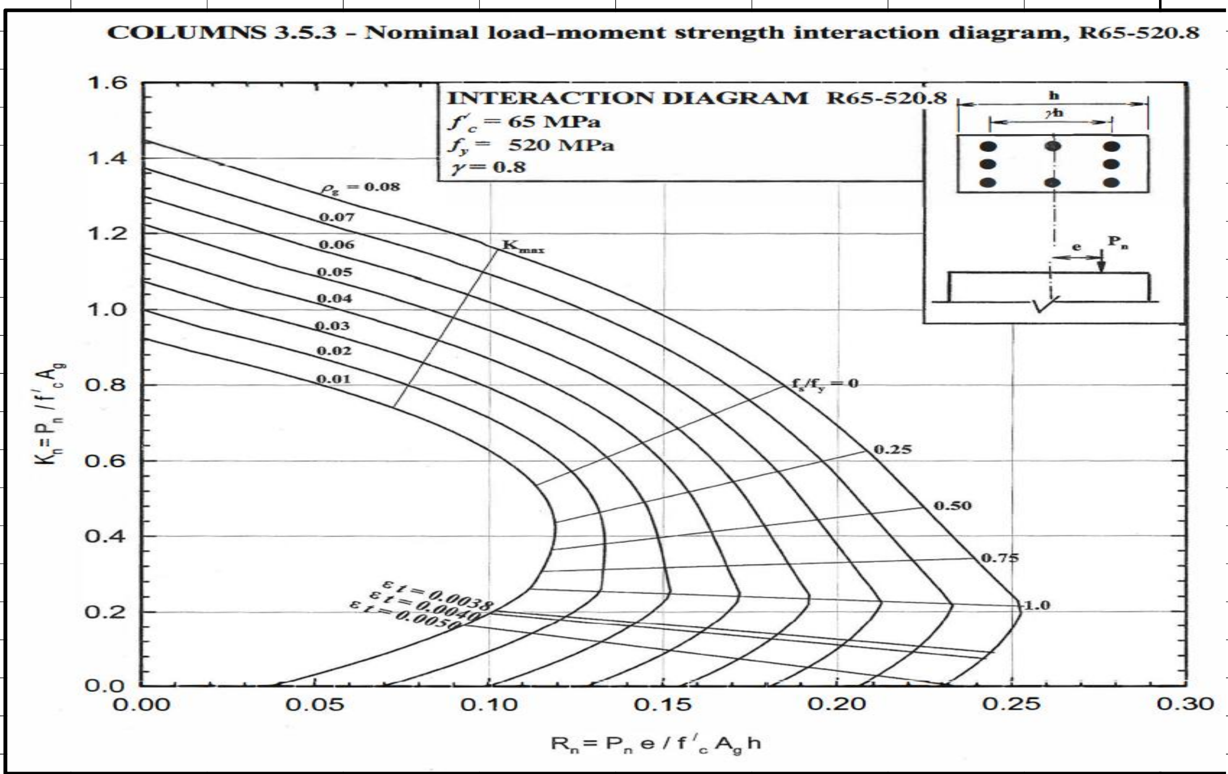
ACI318

Interaction Chart [Major Plane] **Major**



Design nominal axial stress ratio, $\sigma_N/[\phi.f_c']$	ACI318	0.91
Design 1/6th nominal bending stress ratio, $\sigma_{Mx}/[\phi.f_c']$		0.03

Interaction Chart [Minor Plane] **Minor**



Design nominal axial stress ratio, $\sigma_N/[\phi.f_c']$	ACI318	0.91
Design 1/6th nominal bending stress ratio, $\sigma_{My}/[\phi.f_c']$		0.04

CONSULTING ENGINEERS	Engineering Calculation Sheet Consulting Engineers	Job No.	Sheet No.	Rev.
		jXXX	7	
		Member/Location		
Job Title	Member Design - Reinforced Concrete Column Slendern	Drg. Ref.		
Member Design - RC Column Slenderness Effects		Made by XX	Date 26/8/2024	Chd.

BS8110

Step 1: Bracing Condition

	Major	Minor	
Braced or unbraced column ?	Braced	Braced	cl.3.8.1.5

Note braced or unbraced column affects slenderness limits criteria, effective height factor;
Note braced = {column / wall stabilized by other bracing, shear walls or core walls and outriggers};
Note unbraced = {column / wall stabilized by bending in itself (columns in moment frames, tube major plane
Note unbraced cantilever = {column / wall stabilized by bending in itself (shear wall major plane, cantilever p

3.8.1.5 *Braced and unbraced columns*

A column may be considered braced in a given plane if lateral stability to the structure as a whole is provided by walls or bracing or buttressing designed to resist all lateral forces in that plane. It should otherwise be considered as unbraced.

Step 2: Slenderness Limits Criteria Based on Bracing Condition

		Major	Minor		
Max **slender** clear slenderness $l_{clear,x}$/(h|D) | $l_{clear,y}$/(b|D	Braced	60	60		cl.3.8.1.7
	Unbraced	60	60		cl.3.8.1.7
Max **slender** clear height $l_{clear,x}$ | $l_{clear,y}$	Cantilever	174000	27931	mm	cl.3.8.1.8
Max **slender** eff. slenderness $l_{eff,x}$/(h|D) | $l_{eff,y}$/(b|D)	Braced	40	40		cl.3.9.3.7.2
	Unbraced	30	30		cl.3.8.5, cl.3.9.:
Max **stocky** eff. slenderness $l_{eff,x}$/(h|D) | $l_{eff,y}$/(b|D)	Braced	15	15		cl.3.8.1.3
	Unbraced	10	10		cl.3.8.1.3

Step 3: Actual Slenderness

	Major	Minor		
Column effective height, $l_{eff,x\|y} = \beta_{x\|y} \cdot l_{clear,x\|y}$	3,443	3,443	mm	cl.3.8.1.6.1
Column effective height factor, $\beta_{x\|y}$	0.85	0.85		
End condition at top	2	2		
End condition at bot.	2	2		

Table 3.19 — Values of β for braced columns

End condition at top	End condition at bottom		
	1	2	3
1	0.75	0.80	0.90
2	0.80	0.85	0.95
3	0.90	0.95	1.00

Table 3.20 — Values of β for unbraced columns

End condition at top	End condition at bottom		
	1	2	3
1	1.2	1.3	1.6
2	1.3	1.5	1.8
3	1.6	1.8	—
4	2.2	—	—

3.8.1.6.2 *End conditions*

The four end conditions are as follows.

a) *Condition 1.* The end of the column is connected monolithically to beams on either side which are at least as deep as the overall dimension of the column in the plane considered. Where the column is connected to a foundation structure, this should be of a form specifically designed to carry moment.

b) *Condition 2.* The end of the column is connected monolithically to beams or slabs on either side which are shallower than the overall dimension of the column in the plane considered.

c) *Condition 3.* The end of the column is connected to members which, while not specifically designed to provide restraint to rotation of the column will, nevertheless, provide some nominal restraint.

d) *Condition 4.* The end of the column is unrestrained against both lateral movement and rotation (e.g. the free end of a cantilever column in an unbraced structure).

	Major	Minor	
Eff. slenderness $l_{eff,x}$/(h|D) | $l_{eff,y}$/(b|D)	1.2	3.8	
Clear height $l_{clear,x}$ | $l_{clear,y}$	4,050	4,050	mm
Clear slenderness $l_{clear,x}$/(h|D) | $l_{clear,y}$/(b|D)	1.4	4.5	

BS8110

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Step 4: Slenderness Condition [Actual Slenderness vs Slenderness Limits Criteria]

		Major	Minor	
Slender clear slenderness UT, $l_{clear,x}/(h\|D)$ | $l_{clear,y}/(b\|D)$	Braced	2%	8%	cl.3.8.1.7
	Unbraced	N/A	N/A	cl.3.8.1.7
Slender clear height UT, $l_{clear,x}$ | $l_{clear,y}$	Cantilever	N/A	N/A	cl.3.8.1.8
Slender eff. slenderness UT, $l_{eff,x}/(h\|D)$ | $l_{eff,y}/(b\|D)$	Braced	3%	10%	cl.3.9.3.7.2
	Unbraced	N/A	N/A	cl.3.8.5, cl.3.9.3
Stocky eff. slenderness UT, $l_{eff,x}/(h\|D)$ | $l_{eff,y}/(b\|D)$	Braced	8%	26%	cl.3.8.1.3
	Unbraced	N/A	N/A	cl.3.8.1.3

Step 5: Moments From Slenderness Effects

	Major	Minor		
Slenderness condition	Stocky	Stocky		cl.3.8.1.3
Slenderness defl., $\alpha_{u,x\|y} = \beta_a K(h\|D)$ | $\beta_a K(b\|D)$	1	2	mm	cl.3.8.3.1
Slenderness defl. wrt. column dimension	0.02%	0.18%		
Coefficient $\beta_a = 1/2000.(l_{eff,x\|y}/b')^2$, b'={h|D,b|D}	0.001	0.007		cl.3.8.3.1

Table 3.21 — Values of β_a

l_e/b'	12	15	20	25	30	35	40	45	50	55	60
β_a	0.07	0.11	0.20	0.31	0.45	0.61	0.80	1.01	1.25	1.51	1.80

	Value		
Axial loads reduction factor, $K=(N_{uz}-N)/(N_{uz}-N_{bal})\leq 1$	0.25		cl.3.8.3.1
Ultimate axial load, N_{uz} $\boxed{N_{uz} = 0.45 f_{cu} A_c + 0.95 f_y A_{sc}}$	115,750	kN	cl.3.8.3.1
Axial load at balanced failure, $N_{bal}=0.25 f_{cu} A_c$	52,200	kN	cl.3.8.3.1
Add. moments for **slender** columns, $M_{add,x\|y} = N.\alpha_{u,x\|y}$	51 / 163	kNm	cl.3.8.3.1

Step 6: Moments From Imperfection Effects

	Major	Minor		
Imperfection defl., $e_{x\|y}$ = MIN{0.05h|D or 0.05b|D, 20mm}	20	20	mm	cl.3.8.2.4
Imperfection defl. wrt. column dimension	0.69%	2.22%		
Nominal moments from imperfections, $M_{e,x\|y} = N.e_{x\|y}$	2,000	2,000	kNm	cl.3.8.2.4

Step 7: Moments From Primary, Slenderness and Imperfection Effects

	Major	Minor		
Design moment, $M_{x\|y}$ = MAX{|$M_{xp\|yp}$|+$M_{add,x\|y}$, $M_{e,x\|y}$}	5,000	2,000	kNm	cl.3.8.3.2
Design axial stress, σ_N = N/bh | N/D^2	38.3		N/mm^2	
Design 1/6th bending stress, $\sigma_{Mx\|y} = M_x/bh^2$ | M_y/hb^2 | $M_{x\|y}/D^3$	0.7	0.9	N/mm^2	
Eff. depth to dimension ratio, h'/(h|D) | b'/(b|D)	0.88	0.88		

CONSULTING ENGINEERS	Engineering Calculation Sheet Consulting Engineers	Job No.	Sheet No.	Rev.
		jXXX	9	
		Member/Location		
Job Title	Member Design - Reinforced Concrete Column Slendern	Drg. Ref.		
Member Design - RC Column Slenderness Effects		Made by XX	Date 26/8/2024	Chd.

EC2

Step 1: Bracing Condition

	Major	Minor	
Braced or unbraced column ?	Braced	Braced	*cl.5.8.1*
Note braced or unbraced column affects slenderness lin	Single Curvature	Single Curvature	

Note braced = {column / wall stabilized by other bracing, shear walls or core walls and outriggers};

Note unbraced = {column / wall stabilized by bending in itself (columns in moment frames, tube major plane

Note unbraced cantilever = {column / wall stabilized by bending in itself (shear wall major plane, cantilever p

Braced members or systems: structural members or subsystems, which in analysis and design are assumed *not* to contribute to the overall horizontal stability of a structure

Bracing members or systems: structural members or subsystems, which in analysis and design are assumed to contribute to the overall horizontal stability of a structure

Step 2: Slenderness Limits Criteria Based on Bracing Condition

		Major	Minor	
Max **stocky** eff. slenderness, $\lambda_{x\|y} = l_{eff,x}/r_x \mid l_{eff,y}/r_y$		Single Curvature	Single Curvature	
Unbraced and Cant. [C=1.7-(M_1/M_2=1)=0.7] λ_{lim} = 20.A.B.C/√n = 20/(1+0.2φ_{ef}).√(1+2ω).0.7/√n	Unbraced Cantilever	14.5	14.5	*cl.5.8.3.1*
Braced Single Curv. [C=1.7-(M_1/M_2=0)=1.7] λ_{lim} = 20.A.B.C/√n = 20/(1+0.2φ_{ef}).√(1+2ω).1.7/√n	Braced Single	35.1	35.1	*cl.5.8.3.1*
Braced Double Curv. [C=1.7-(M_1/M_2=-1)=2.7] λ_{lim} = 20.A.B.C/√n = 20/(1+0.2φ_{ef}).√(1+2ω).2.7/√n	Braced Double	55.8	55.8	*cl.5.8.3.1*

Step 3: Actual Slenderness

		Major	Minor		
Column effective height, $l_{eff,x\|y} = \beta_{x\|y}.l_{clear,x\|y}$		4,029	3,848	mm	*cl.5.8.3.2*
Column effective height factor, $\beta_{x\|y}$		*0.99*	*0.95*		*cl.5.8.3.2*
β=0.5.√[(1+k₁/(0.45+k₁)).(1+k₂/(0.45+k₂)]	Braced	*0.99*	*0.95*		
β=√[1+10.(k₁.k₂)/(k₁+k₂)], k₂=0 for cant.	Unbraced	*14.55*	*4.61*		
β=(1+k₁/(1+k₁)).(1+k₂/(1+k₂)), k₂=0 for ca	Unbraced	*3.91*	*3.25*		

Braced members (see Figure 5.7 (f)):

$$l_0 = 0{,}5l \cdot \sqrt{\left(1+\frac{k_1}{0{,}45+k_1}\right)\cdot\left(1+\frac{k_2}{0{,}45+k_2}\right)} \qquad (5.15)$$

Unbraced members (see Figure 5.7 (g)):

$$l_0 = l \cdot \max\left\{\sqrt{1+10\cdot\frac{k_1\cdot k_2}{k_1+k_2}}\;;\;\left(1+\frac{k_1}{1+k_1}\right)\cdot\left(1+\frac{k_2}{1+k_2}\right)\right\} \qquad (5.16)$$

Unbraced cantilever

k_2 = 0 (fixed end)

where:

k_1, k_2 are the relative flexibilities of rotational restraints at ends 1 and 2 respectively:

$\frac{1}{4}\frac{(I/l_{column})}{(I/l_{beam})} = k$	*0 (fixed end)*	*0.0625*	*0.125*	*0.25*	*0.50*	*1.0*	*1.5*	*2.0*
l_0 – braced (equation 9.2) {×l}	0.5	0.56	0.61	0.68	0.76	0.84	0.88	0.91
l_0 – unbraced (equation 9.3(a) and	1.0	1.14	1.27	1.50	1.87	2.45	2.92	3.32
9.3(b)). Use greater value {×l}	1.0	1.12	1.13	1.44	1.78	2.25	2.56	2.78

	Major	Minor
Eff. slenderness $\lambda_{x\|y} = l_{eff,x}/r_x \mid l_{eff,y}/r_y$	4.8	14.8

CONSULTING ENGINEERS	Engineering Calculation Sheet Consulting Engineers	Job No.	Sheet No.	Rev.
		jXXX	10	
		Member/Location		
Job Title	Member Design - Reinforced Concrete Column Slendern	Drg. Ref.		
Member Design - RC Column Slenderness Effects		Made by XX	Date 26/8/2024	Chd.

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Step 4: Slenderness Condition [Actual Slenderness vs Slenderness Limits Criteria]

		Major	Minor	
		Single Curvature	Single Curvature	
Stocky eff. slenderness UT, $\lambda_{x\|y} = l_{eff,x}/r_x \mid l_{eff,y}/r_y$	Unbraced Cantilever	N/A	N/A	cl.5.8.3.1
Stocky eff. slenderness UT, $\lambda_{x\|y} = l_{eff,x}/r_x \mid l_{eff,y}/r_y$	Braced Single	14%	42%	cl.5.8.3.1
Stocky eff. slenderness UT, $\lambda_{x\|y} = l_{eff,x}/r_x \mid l_{eff,y}/r_y$	Braced Double	N/A	N/A	cl.5.8.3.1

Step 5: Moments From Slenderness Effects

	Major	Minor		
Slenderness condition	Stocky	Stocky		cl.5.8.3.1
Slenderness defl., $e_{2,x\|y} = (1/r).l_{eff,x\|y}{}^2/c$	1	3	mm	cl.5.8.8.2
Slenderness defl. wrt. column dimension	0.04%	0.32%		
$\omega = A_{sc}/A_c.f_{yd}/f_{cd}$, $f_{yd}=f_y/1.15$, $f_{cd}=\alpha_{cc}.f_{ck}/\gamma_c=0.85.f_{ck}/1.5$	0.226			cl.5.8.3.1
$\varphi_{ef} = \varphi(\infty,t_0).\{M_{SLS}/M_{ULS}=1/1.4\}$	0.71			cl.5.8.4
$\varphi(\infty,t_0)$	1.00			cl.3.1.4
$n = N_{Ed}/(A_c.f_{cd})$, $f_{cd}=\alpha_{cc}.f_{ck}/\gamma_c=0.85.f_{ck}/1.5$	1.04			cl.5.8.3.1
$c = \pi^2$	9.87			cl.5.8.8.2
$1/r = K_r.K_\varphi.1/r_0$	0.062	0.194	$/10^5$mm	cl.5.8.8.3
$K_r = (n_u-n)/(n_u-n_{bal}) \leq 1$, $n_u=1+\omega$, $n_{bal}=0.4$	0.22			cl.5.8.8.3
$K_\varphi = 1+\beta\varphi_{ef} \geq 1$, $\beta=0.35+f_{ck}/200-\lambda_{x\|y}/150$	1.46	1.41		cl.5.8.8.3
$1/r_0 = \varepsilon_{yd}/(0.45h'\|b') = f_{yd}/E_s/(0.45h'\|b')$ = f	0.189	0.613	$/10^5$mm	cl.5.8.8.3
Add. moments for **slender** columns, $M_{add,x\|y} = N.e_{2,x\|y}$	102	291	kNm	cl.5.8.8.2

Step 6: Moments From Imperfection Effects

	Major	Minor		
Imperfection defl., $e_{x\|y}$ = MAX{h|D/30 or b|D/30, 20mm}	97	30	mm	cl.6.1
Imperfection defl. wrt. column dimension	3.33%	3.33%		
Nominal moments from imperfections, $M_{e,x\|y} = N.e_{x\|y}$	9,667	3,000	kNm	cl.6.1

Step 7: Moments From Primary, Slenderness and Imperfection Effects

	Major	Minor		
Design moment, $M_{x\|y} = MAX\{\|M_{xp\|yp}\|+N.e_{a,x\|y}+M_{add,x\|y}, M_{e,x\|y}\}$	9,667	3,000	kNm	cl.5.8.8.2
Accidental imperfection defl., $e_{a,x\|y} = l_{eff,x\|y}/400$	10	10	mm	5.2, cl.5.8.8
Design axial stress, $\sigma_N = N/bh \mid N/D^2$	38.3		N/mm^2	
Design 1/6th bending stress, $\sigma_{Mx\|y} = M_x/bh^2 \mid M_y/hb^2 \mid M_{x\|y}/D^3$	1.3	1.3	N/mm^2	
Eff. depth to dimension ratio, h'/(h|D) | b'/(b|D)	0.88	0.88		

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Step 1: Bracing Condition

	Major	Minor	
Braced or unbraced column (non-sway or sway) ?	Braced	Braced	cl.6.2.5
Note braced or unbraced column affects slenderness lin	Single Curvature	Single Curvature	*erness mag*

Note braced = {column / wall stabilized by other bracing, shear walls or core walls and outriggers};

Note unbraced = {column / wall stabilized by bending in itself (columns in moment frames, tube major plane

Note unbraced cantilever = {column / wall stabilized by bending in itself (shear wall major plane, cantilever p

Braced check method 1:	$\Sigma I_{braced\_walls/columns} \leq 1/12^{th}\ \Sigma I_{bracing\_walls}$	**Note**	*cl.6.2.5*
Braced check method 2:	$\Sigma V_{braced\_walls/columns} \leq 1/12^{th}\ \Sigma V_{bracing\_walls}$	**Note**	*cl.6.2.5*
Braced check method 2:	$\Sigma M_{braced\_walls/columns} \leq 1/12^{th}\ \Sigma M_{bracing\_walls}$	**Note**	*cl.6.2.5*
Braced check method 3:	$Q \leq 0.25$ or $\lambda \geq 4.0$, but with P-Δ	**Note**	*cl.6.2.6*
Braced check method 4:	$Q \leq 0.05$ or $\lambda \geq 20$, and without P-Δ	**Note**	*cl.6.6.4.3(b*

Step 2: Slenderness Limits Criteria Based on Bracing Condition

		Major	Minor	
Max **stocky** eff. slenderness, $\lambda_{x\|y} = l_{eff,x}/r_x \mid l_{eff,y}/r_y$		Single Curvature	Single Curvature	
Unbraced and Cant. λ_{lim} = 22	**Unbraced** **Cantilever**	22.0	22.0	cl.6.2.5
Braced Single Curv. [M_1/M_2=0] λ_{lim} = MIN{40, 34-12M_1/M_2} = 34	**Braced Single**	34.0	34.0	cl.6.2.5
Braced Double Curv. [M_1/M_2=–1] λ_{lim} = MIN{40, 34-12M_1/M_2} = 40	**Braced Double**	40.0	40.0	cl.6.2.5

Step 3: Actual Slenderness

		Major	Minor		
Column effective height, $l_{eff,x\|y} = \beta_{x\|y}.l_{clear,x\|y}$		**4,029**	**3,848**	mm	cl.R6.2.5
Column effective height factor, $\beta_{x\|y}$		***0.99***	***0.95***		cl.R6.2.5
β=0.5.√[(1+k₁/(0.45+k₁)).(1+k₂/(0.45+k₂)]	**Braced**	0.99	0.95		cl.R6.2.5
β=√[1+10.(k₁.k₂)/(k₁+k₂)], k₂=0 for cant.	**Unbraced**	14.55	4.61		cl.R6.2.5
β=(1+k₁/(1+k₁)).(1+k₂/(1+k₂)), k₂=0 for ca	**Unbraced**	3.91	3.25		cl.R6.2.5

Braced members (see Figure 5.7 (f)):

$$l_0 = 0{,}5l \cdot \sqrt{\left(1+\frac{k_1}{0{,}45+k_1}\right)\cdot\left(1+\frac{k_2}{0{,}45+k_2}\right)} \quad (5.15)$$

Unbraced members (see Figure 5.7 (g)):

$$l_0 = l \cdot \max\left\{\sqrt{1+10\cdot\frac{k_1\cdot k_2}{k_1+k_2}}\ ;\ \left(1+\frac{k_1}{1+k_1}\right)\cdot\left(1+\frac{k_2}{1+k_2}\right)\right\} \quad (5.16)$$

Unbraced cantilever

k_2 = 0 (fixed end)

where:

k_1, k_2 are the relative flexibilities of rotational restraints at ends 1 and 2 respectively:

$\frac{1}{4}\frac{(I/l_{column})}{(I/l_{beam})} = k$	0 (fixed end)	0.0625	0.125	0.25	0.50	1.0	1.5	2.0
l_0 – braced (equation 9.2) $\{\times l\}$	0.5	0.56	0.61	0.68	0.76	0.84	0.88	0.91
l_0 – unbraced (equation 9.3(a) and	1.0	1.14	1.27	1.50	1.87	2.45	2.92	3.32
9.3(b)). Use greater value $\{\times l\}$	1.0	1.12	1.13	1.44	1.78	2.25	2.56	2.78

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	Major	Minor
Eff. slenderness $\lambda_{x\|y} = l_{eff,x}/r_x \mid l_{eff,y}/r_y$	**4.8**	**14.8**

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Step 4: Slenderness Condition [Actual Slenderness vs Slenderness Limits Criteria]

		Major	Minor	
		Single Curvature	Single Curvature	
Stocky eff. slenderness UT, $\lambda_{x\|y}$ = $l_{eff,x}/r_x$ | $l_{eff,y}/r_y$	Unbraced Cantilever	N/A	N/A	*cl.6.2.5*
Stocky eff. slenderness UT, $\lambda_{x\|y}$ = $l_{eff,x}/r_x$ | $l_{eff,y}/r_y$	Braced Single	14%	44%	*cl.6.2.5*
Stocky eff. slenderness UT, $\lambda_{x\|y}$ = $l_{eff,x}/r_x$ | $l_{eff,y}/r_y$	Braced Double	N/A	N/A	*cl.6.2.5*

Step 5: Moments From Slenderness Effects

	Major	Minor		
Slenderness condition	Stocky	Stocky		
Magnification factor, $\delta_{x\|y}$ = MAX{1.0, C_m/[1-N/(0.75P_c)]}	1.00	1.00		*.4.5, cl.6.6.*
Slenderness defl. wrt. column dimension	0.00%	0.00%		
Correction factor, C_m	0.6	0.6		*cl.6.6.4.5.3*
Note for braced single curv., C_m = 0.6+0.4.[M_1/M_2=0], and thus C_m = 0.6;				*cl.6.6.4.5.3*
Note for braced double curv., C_m = 0.6+0.4.[$M_1/M_2 = -1$], and thus C_m = 0.2;				*cl.6.6.4.5.3*
Note for unbraced, C_m = 1.0 (by equalising storey and individual column effects);				*cl.6.6.4.6.2*
Euler buckling, $P_c = \pi^2.(EI)_{eff}/l_{eff,x\|y}{}^2$	10,538	1,113	MN	*cl.6.6.4.4.2*
$(EI)_{eff} = 0.4EI_{x\|y}/(1+\beta_d)$, β_d=0.6 (braced), β_d	17,328	1,669	10^3kNm^2	*4.4.4, cl.6.6*
Add. moments for **slender** columns, $M_{add,x\|y} = (\delta_{x\|y}-1).\|M_{2,x\|y}\|$	0	0	kNm	*4.5.1, cl.6.6*

Step 6: Moments From Imperfection Effects

	Major	Minor		
Imperfection defl., $e_{x\|y}$ = [0.03h|D or 0.03b|D] + 15mm	102	42	mm	*cl.6.6.4.5.4*
Imperfection defl. wrt. column dimension	3.52%	4.67%		
Nominal moments from imperfections, $M_{e,x\|y} = N.e_{x\|y}$	10,200	4,200	kNm	*cl.6.6.4.5.4*

Step 7: Moments From Primary, Slenderness and Imperfection Effects

	Major	Minor		
Design moment, $M_{x\|y}$ = MAX{|$M_{xp\|yp}$|+$M_{add,x\|y}$, $M_{e,x\|y}$}	10,200	4,200	kNm	*cl.6.6.4.6.1*
Design axial stress, σ_N = N/bh | N/D^2	38.3		N/mm^2	
Design 1/6th bending stress, $\sigma_{Mx\|y}$ = M_x/bh^2 | M_y/hb^2 | $M_{x\|y}/D^3$	1.3	1.8	N/mm^2	
Eff. depth to dimension ratio, h'/(h|D) | b'/(b|D)	*0.88*	*0.88*		