CONSULTING ENGINEERS	Engineering Calculation Sheet Consulting Engineers	Job No. jXXX	Sheet No. 1	Rev.
		Member/Location		
Job Title	Member Design - Reinforced Concrete Column BS8110	Drg. Ref.		
Member Design - RC Column		Made by XX	Date 22/7/2024	Chd.

				BS8110
Effects From Structural Analysis				
Axial force, N (tension -ve and comp +ve) (ensure >= 0)	100000	kN		OK
Major plane shear force, V_y	0	kN		
Minor plane shear force, V_z	0	kN		
Major plane primary bending moment, M_{xp}	5000	kNm		
Minor plane primary bending moment, M_{yp}	1000	kNm		
Imperfection deflection (in h direction), e_h = MIN (0.05h, 20mm)	20	mm		cl.3.8.2.4
Imperfection deflection (in b direction), e_b = MIN (0.05b, 20mm)	20	mm		cl.3.8.2.4
Major plane imperfection (nominal) moment, $M_{eh} = N.e_h$	2000	kNm		cl.3.8.2.4
Minor plane imperfection (nominal) moment, $M_{eb} = N.e_b$	2000	kNm		cl.3.8.2.4
Major plane max design bending moment, M_x = MAX ($M_{xp}+M_{add,x}$, M_{eh})	**5000**	kNm		cl.3.8.3.2
Minor plane max design bending moment, M_y = MAX ($M_{yp}+M_{add,y}$, M_{eb})	**2000**	kNm		cl.3.8.3.2
Material Properties				
Characteristic strength of concrete, f_{cu} (≤ 105N/mm^2; HSC)	80	N/mm^2		OK
Yield strength of longitudinal steel, f_y	460	N/mm^2		
Yield strength of shear link steel, f_{yv}	460	N/mm^2		
Bracing or Unbraced Column				
	Major	**Minor**		
Braced or unbraced column ? (affects slenderness limits criteria	Braced	Braced		cl.3.8.1.5
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 or tube major plan				
Section Dimensions				
Section type (affects concrete area, slenderness, steel area req)	Rectangular			
Depth (larger), h (rectangular) or diameter, D (circular)	2900	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)	2610000	mm^2		
Major plane clear height, $l_{clear,x}$	4.050	m		cl.3.8.1.6
Minor plane clear height, $l_{clear,y}$	4.050	m		cl.3.8.1.6
Major plane effective height, $l_{eff,x}$	3.443	m		cl.3.8.1.6
Minor plane effective height, $l_{eff,y}$	3.443	m		cl.3.8.1.6
Longitudinal steel reinforcement diameter, ϕ	32	mm		
Total longitudinal steel reinforcement number (uniaxial bending), n_l	50			Note
Total longitudinal steel area provided (uniaxial bending), $A_{sc} = n_l.\pi.\phi^2/4$	**40212**	mm^2		
Total longitudinal steel reinforcement number (orthogonal bending), n_{l+}	12			Note
Total longitudinal steel area provided (orthogonal bending), $A_{sc+} = n_{l+}.\pi.\phi^2/4$	**9651**	mm^2		
Total longitudinal steel area provided, $A_{sc}+A_{sc+}$	**49863**	mm^2		
(Note A_{sc} is the total longitudinal steel area for the relevant uniaxial plane of bending only, whilst A_{sc+}				
is the total longitudinal steel area for bending in the orthogonal plane, excluding steel counted within A_{sc})				
Shear link diameter, ϕ_{link}	12	mm		
Number of links in a cross section, i.e. number of legs, n_v	6			
Area provided by all links in a cross-section, $A_{sv,prov} = n_v.\pi.\phi_{link}^2/4$	**679**	mm^2		
Pitch of links, S	100	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		

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Utilisation Summary

	Major	Minor
Braced or unbraced	Braced	Braced
	Major	Minor
Slenderness (short or slender)	Short	Short

Item	UT	Remark
Max (braced) slenderness	10%	OK
Max (unbraced) slenderness / height	N/A	N/A
Shear ultimate stress	0%	OK
Shear (with axial load) design capacity	79%	OK
Shear (axial confinement) design capacity	89%	OK
Method 1 (nominal moments; slender column Euler buc	4%	OK
Method 2 (nominal moments; short column crushing)	101%	NOT OK
Method 3 (small assumed moments; short column crusl	115%	NOT OK
Method 4 (biaxial design moments; short column crushi	93%	OK
Total utilisation	**115%**	**NOT OK**
Detailing requirements	**OK**	

Convergence
Converged

Design Column (Iterative)

% Vertical reinforcement	**1.91**	%
Estimated steel reinforcement quantity (220 – 300kg/m^3)	**228**	kg/m^3
7850 . [(A_{sc} +A_{sc+}) / A_c + ($A_{sv,prov}$.(h+b or 2D)/S) / A_c]; No laps;		
Estimated steel reinforcement quantity (220 – 300kg/m^3)	**319**	kg/m^3 *IStructE*
11000 . [(A_{sc} +A_{sc+}) / A_c + ($A_{sv,prov}$.(h+b or 2D)/S) / A_c]; Laps;		

e, shear w *[Note that steel quantity in kg/m^3 can be obtained from 110.0 x % rebar];*

Material cost:	concrete, c	250	units/m^3	steel, s	3500 units/tonne
Reinforced concrete material cost = [c+(est. rebar quant).s].A_c					**3565** units/m

Column Effective Height

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).

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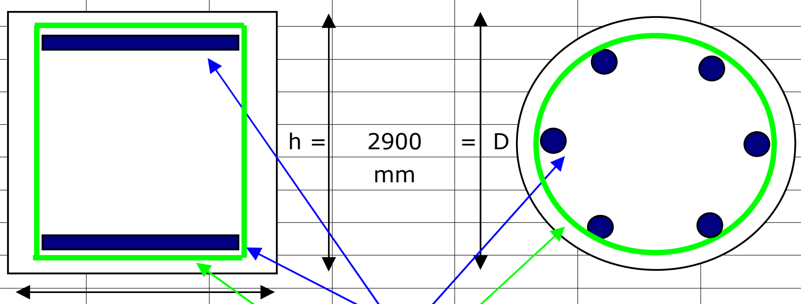
Effective Depth and Width

Number of layers of steel at each extremity for rect cols, n_{layers}			1	layer(s)
(Note n_{layers} affects the effective h' or b' depending on equivalent single axis of bending, for rect only)				
Spacer reinforcement, s_r = MAX (ϕ, 25mm, user)	150	mm	150	mm
Plane of bending	**b-plane**	or	**minor plane**	
Effective depth, h' = h - $cover_{main}$ - [ϕ+(n_{layers}-1)(ϕ+s_r)]/2 rect		**98%**	2847	mm
= D - $cover_{main}$ - ϕ/2 circular				
Effective width, b' = b - $cover_{main}$ - [ϕ+(n_{layers}-1)(ϕ+s_r)]/2 rect		**94%**	847	mm
= D - $cover_{main}$ - ϕ/2 circular				

(Note multiple steel layer for h'- or b'- plane bending depending on equivalent single axis of bending, for rect o

Detailing Instructions



b = 900 mm

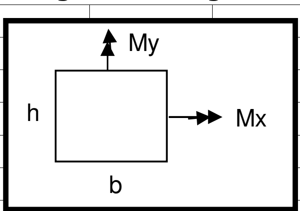
A_{sc} = 50 T32 Symmetrically Distributed

Links = 6 legs of T12@100mm pitch

Cover =	25 mm
Concrete =	80 MPa
Rebars =	460 MPa
Links =	460 MPa
Steel % =	1.91 %
Bending plane =	b-plane
n_{layers} =	1

(Note rect column shown for bending in h-plane, not b-plane)

Bending Moment Sign Convention



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		Member/Location		
Job Title	Member Design - Reinforced Concrete Column BS8110	Drg. Ref.		
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				BS8110
Slenderness of Column (Whether Short or Slender)				
Major plane eff. slenderness, $l_{eff,x}$/(h or D)	Major	1.2		
Minor plane eff. slenderness, $l_{eff,y}$/(b or D)	Minor	3.8		
Major plane short column limiting eff. slenderness (15 braced; 10 u	Major	15.0		cl.3.8.1.3
Minor plane short column limiting eff. slenderness (15 braced; 10 u	Minor	15.0		cl.3.8.1.3
Major plane eff. slenderness (short if < criteria, slender if > criteria	Major	Short		cl.3.8.1.3
Minor plane eff. slenderness (short if < criteria, slender if > criteria	Minor	Short		cl.3.8.1.3
Major plane max clear slenderness $l_{clear,x}$/(h or D)	Major	1.4		cl.3.8.1.7
Minor plane max clear slenderness $l_{clear,y}$/(b or D)	Minor	4.5		cl.3.8.1.7
Max (braced or unbraced) clear slenderness utilisation (<= 60)		8%		OK
Major plane max eff. slenderness $l_{eff,x}$/(h or D)	Major	1.2		cl.3.9.3.7.2
Minor plane max eff. slenderness $l_{eff,y}$/(b or D)	Minor	3.8		cl.3.9.3.7.2
Max (braced) eff. slenderness utilisation (<= 40)		10%		OK
Major plane max clear height $l_{clear,x}$	N/A	4050	mm	cl.3.8.1.8
Minor plane max clear height $l_{clear,y}$	N/A	4050	mm	cl.3.8.1.8
Max (unbraced cant.) clear height utilisation (<= 60(h or b) or 100(h or b)^2/(l		N/A		N/A
Major plane max eff. slenderness $l_{eff,x}$/(h or D)	Major	1.2		cl.3.8.5, cl.3.9.3
Minor plane max eff. slenderness $l_{eff,y}$/(b or D)	Minor	3.8		cl.3.8.5, cl.3.9.3
Max (unbraced) eff. slenderness utilisation (<= 30)		N/A		N/A
Note for RC columns and walls, slenderness limits are as follows:-				
braced short (stocky) $l_{eff,x/y}$/(h/b or D)		*15*		*cl.3.8.1.3*
braced slender $l_{clear,x/y}$/(h/b or D)		*60*		*cl.3.8.1.7*
braced slender $l_{eff,x/y}$/(h/b or D)		*40*		*cl.3.9.3.7.2*
unbraced short (stocky) $l_{eff,x/y}$/(h/b or D)		*10*		*cl.3.8.1.3*
unbraced slender $l_{clear,x/y}$/(h/b or D)		*60*		*cl.3.8.1.7*
unbraced cant. slender $l_{clear,x/y}$	*{60h, 100h^2/b} | {60b, 100b^2/h}*			*cl.3.8.1.8*
unbraced slender $l_{eff,x/y}$/(h/b or D)		*30*		*cl.3.8.5*
unbraced slender $l_{eff,x/y}$/(h/b or D)		*30*		*cl.3.9.3.7.2*
Note for plain (unreinforced) walls, slenderness limits are as follows:-				
braced short (stocky) l_{eff}/THK		*15*		*cl.3.8.1.3*
unbraced short (stocky) l_{eff}/THK		*10*		*cl.3.8.1.3*
braced or unbraced slender l_{eff}/THK		*30*		*cl.3.9.4.4*

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Moments From Slenderness Effects				cl.3.8.3.1
Additional moment for slender columns, $M_{add,x}$	$\boxed{M_{add} = Na_u}$	**N/A**	kNm	cl.3.8.3.1
Additional moment for slender columns, $M_{add,y}$		**N/A**	kNm	cl.3.8.3.1
Major plane effective height, $l_{eff,x}$		N/A	m	cl.3.8.3.1
Minor plane effective height, $l_{eff,y}$		N/A	m	cl.3.8.3.1
Deflection in x (h in this equation = h or D)	$\boxed{a_u = \beta_a K h}$	N/A	mm	cl.3.8.3.1
Deflection in y (h in this equation = b or D)		N/A	mm	cl.3.8.3.1
Coefficient in x (b' in this equation = h or D)	$\boxed{\beta_a = \frac{1}{2000}\left(\frac{l_e}{b'}\right)^2}$	N/A		cl.3.8.3.1
Coefficient in y (b' in this equation = b or D)		N/A		cl.3.8.3.1
	$\boxed{K = \frac{N_{uz} - N}{N_{uz} - N_{bal}} \leq 1}$			
Reduction factor due to axial loads		0.25		cl.3.8.3.1
Ultimate axial load	$\boxed{N_{uz} = 0.45 f_{cu} A_c + 0.95 f_y A_{sc}}$	115750	kN	cl.3.8.3.1
Axial load at balanced failure, $N_{bal} = 0.25 f_{cu} A_c$		52200	kN	cl.3.8.3.1
Single Axis Moment From Biaxial Moments				
Major plane max design bending moment, M_x		**5000**	kNm	
Minor plane max design bending moment, M_y		**2000**	kNm	
Ratio $N/(bhf_{cu})$ rectangular or $N/(D^2 f_{cu})$ circular		0.48		cl.3.8.4.5
Enhancement coefficient for biaxial bending, β		0.44		cl.3.8.4.5

Table 3.22 — Values of the coefficient β

$\frac{N}{bhf_{cu}}$	0	0.1	0.2	0.3	0.4	0.5	≥0.6
β	1.00	0.88	0.77	0.65	0.53	0.42	0.30

Effective depth, h' = h or D - $cover_{main}$ - ϕ/2		2847	mm	
Effective width, b' = b or D - $cover_{main}$ - ϕ/2		847	mm	
(Note for the purpose of determining equivalent single bending axis, single steel layer assumed)				
If $\boxed{M_x/h' \geq M_y/b'}$ then increased major plane bending	$\boxed{M_x + \beta \frac{h'}{b'} M_y}$	**N/A**	kNm	cl.3.8.4.5
If $\boxed{M_x/h' < M_y/b'}$ then increased minor plane bending	$\boxed{M_y + \beta \frac{b'}{h'} M_x}$	**2659**	kNm	cl.3.8.4.5
Increased single axis bending moment, M		**2659**	kNm	cl.3.8.4.5
Plane of design moment for rectangular columns (h- or b-)		**b-plane**		cl.3.8.4.5

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Shear (With Axial Load)				*cl.3.4.5.12*
Shear insignificant if M/N < 0.6 (h or b) for rect, 0.6 D for circ	27 <	540	mm	*cl.3.8.4.6*
(Note h or b depending on equivalent single axis of ben	**Shear Insignificant**			*cl.3.8.4.6*
Maximum shear force, V_d = MAX (V_y, V_z)		0	kN	
Ultimate shear stress, $v_{ult} = V_d / A_c$ (< $0.8f_{cu}^{0.5}$ & {5.0,7.0}N/mm^2)		**0.00**	N/mm^2	*.3.4.5.2 BC*
Note the ultimate shear stress limit of 5.0 or 7.0N/mm^2 is used for f_{cu} ≤ 60 or 105N/mm^2 respectively;				
Ultimate shear stress utilisation		**0%**		**OK**
Design shear stress, $v_d = V_d / A_c$		**0.00**	N/mm^2	
(Shear capacity enhancement by ***either*** *calculating v_d at d from support and comparing against*				
unenhanced v_c as clause 3.4.5.10 BS8110 ***or*** *calculating v_d at support and comparing against*				
enhanced v_c within 2d of the support as clause 3.4.5.8 BS8110 both ***not applicable*** *as described*				
in clause 3.4.5.12 BS8110;)				
Area of tensile steel reinforcement provided (uniaxial bending), $A_{s,prov} = A_{sc}$ / 2		20106	mm^2	
$\rho_w = 100A_{s,prov}/A_c$		0.77	%	
Effective distance to tension steel, h' or b'		847	mm	
(Note h' or b' depending on equivalent single axis of bending, for rect only)				
$v_c = (0.79/1.25)(\rho_w f_{cu}/25)^{1/3}(400/(h' \text{ or } b'))^{1/4}$; ρ_w<3; f_{cu}<80; (400/(h' or b'))^1		**0.85**	N/mm^2	*.3.4.5.4 BC*
Including axial force effects	$v_c' = v_c + 0.6\dfrac{NVh}{A_c M}$ < $v_c' = v_c\sqrt{\{1+N/(A_c v_c)\}}$	**0.85**	N/mm^2	*cl.3.4.5.12*
N/A_c		38.3	N/mm^2	*cl.3.4.5.12*
V_d(h or b)/M or V_dD/M but < 1.0		0.00		*cl.3.4.5.12*
(Note h or b depending on equivalent single axis of bending, for rect only)				
Minimum shear strength, v_r = MAX (0.4, $0.4(f_{cu}/40)^{2/3}$), f_{cu}≤80N/mm^2		**0.63**	N/mm^2	*.3.4.5.3 BC*
Check v_d < 0.5v_c' (column) (minor elements) or 1.0v_c' (wall) for no lin		**VALID**	**Column**	*cl.3.8.4.6*
Concrete shear capacity $v_c'.(A_c)$		**2228**	kN	
Check 0.0v_c' (column) or 1.0v_c' (wall) < v_d < v_r + v_c' for nominal links		**VALID**		*cl.3.4.5.3*
$(A_{sv}/S)_{nom}$ > v_r.(b or h rect, D circ)/(0.95f_{yv}) i.e. $(A_{sv}/S)_{nom}$ >		**4.21**	mm^2/mm	
(Note b or h depending on equivalent single axis of bending, for rect only)				
$V_{cap,nom} = (v_r + v_c').(A_c)$		**3886**	kN	
Check v_d > v_r + v_c' for design links		**N/A**		*cl.3.4.5.3*
A_{sv}/S > (b or h rect, D circ)(v_d-v_c')/(0.95f_{yv}) i.e. A_{sv}/S >		**4.21**	mm^2/mm	
(Note b or h depending on equivalent single axis of bending, for rect only)				
$V_{cap} = (A_{sv,prov}/S).(0.95f_{yv})$.(h or b rect, D circ) + $v_c'.(A_c)$		**4897**	kN	
Area provided by all links in a cross-section, $A_{sv,prov}$		679	mm^2	
Tried $A_{sv,prov}$ / S value		6.79	mm^2/mm	
Design shear (with axial load) resistance utilisation		**79%**		**OK**
Shear (Axial Confinement)	Consider for Columns Only			
Minimum confining pressure, f_s	Non-Seismic Design | 0.015fck	0.98	N/mm^2	*McFarlane*
				IStructE, 07
Confining pressure, $f_s = [A_{sv,prov}/S].f_{yv}/b_c$		1.10	N/mm^2	*McFarlane*
Width, b_c = [(b or h) for rect, 0.6 D for circ] – 2.cover – ϕ_{link}		2838	mm	*StructE, 07*
Area provided by all links in a cross-section, $A_{sv,prov}$		679	mm^2	
Tried $A_{sv,prov}$ / S value		6.79	mm^2/mm	
Design shear (axial confinement) resistance utilisation		**89%**		**OK**

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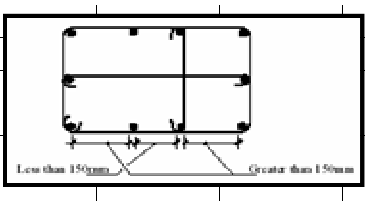
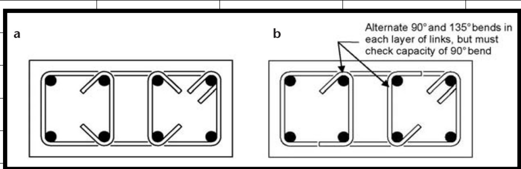
Detailing Requirements

All detailing requirements met ?	**OK**		
By definition, b <= h			**OK**
Min dimension (to facilitate concreting >= 125mm)	**900**	mm	**OK**
Min longitudinal steel reinforcement number, n_l (>= 4 rectangular; >=6 circul	**50**		**OK**
Min longitudinal steel reinforcement diameter, ϕ (>=12mm column **Column**	**32**	mm	**OK**
Percentage of reinforcement $(A_{sc}+A_{sc+})/A_c$ x 100%	**1.91**	%	**OK**
Percentage of reinforcement A_{sc}/A_c x 100% (>0.40%, [0.40+0.01(f_{cu}-60)]% and <5.00%)			*TR49 cl.3.1.*
Longitudinal steel reinforcement pitch (>75mm+ϕ, >100mm+ϕ if T40; <=300	**116**	mm	**OK**
Rectangular col bar pitch = [(b or h)-2.cover$_{main}$-ϕ]/(n_l/(2.n_{layers})-1)	116	mm	
(Note b or h depending on equivalent single axis of bending, for rect only)			
Circular col bar pitch = π.(D-2.cover$_{main}$-ϕ)/n_l	N/A	mm	
Note an allowance has been made for laps in the min pitch by increasing the criteria by the bar diameter.			
Min link diameter, ϕ_{link} (>=0.25ϕ; >=6mm NSC; >=10mm HSC)	**12**	mm	**OK**
Max link pitch, S	**100**	mm	**OK**
Max link pitch, S (<=12 ϕ NSC, <=10 ϕ HSC, <=24 ϕ_{link} HSC, <=300mm, <=	*288*	*mm*	
Require an overall enclosing link.			
Require additional restraining links for each alternate longitudinal bar in each direction.			
No unrestrained bar should be further than 150mm clear distance from a restrained bar.			



2

Require through slab / beam depth column links in edge and corner columns due to lack of restraint.			
Max link pitch, S	**100**	mm	**OK**
Max link pitch, S (<=10 ϕ.f_1.f_2.f_3 HSC, <=24 ϕ_{link}.f_1.f_2.f_3 HSC)	*149*	*mm*	*McFarlane*
Axial stress, N/(f_{cu}.A_c)	0.48		*IStructE, 07*
Spacing factor, f_1 = 0.27(f_{cu}.A_c)/N	0.56		
Spacing factor, f_2 = ϕ_{link}/12	1.00		
Spacing factor, f_3 = f_{yv}/500	0.92		



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Method 1 (Axial Force; Nominal Moments for Non-Continuous (Precast) Floors; Slender Column Eule

Axial buckling capacity (Euler) major plane, $N_{cap,Euler} = \pi^2.E_{c,28}.I_x/l_{eff,x}^2$	**3E+07**	kN	
Axial buckling capacity (Euler) minor plane, $N_{cap,Euler} = \pi^2.E_{c,28}.I_y/l_{eff,y}^2$	**3E+06**	kN	
Elastic modulus of concrete, $\boxed{E_{c,28} = K_o + 0.2 f_{cu,28}}$	36.0	GPa	
Cracked second moment of area major plane, $0.5I_x = 0.5b.h^3/12$ o	9.1E+11	mm^4	
Cracked second moment of area minor plane, $0.5I_y = 0.5h.b^3/12$ o	8.8E+10	mm^4	
Axial capacity utilisation = $N/N_{cap,Euler}$	**4%**		**OK**

Method 2 (Axial Force; Nominal Moments for Non-Continuous (Precast) Floors; Short Column Crushi

Percentage of reinforcement $(A_{sc}+A_{sc+})/A_c$ x 100%	1.91	%	
Axial capacity, $N_{cap} = 0.40f_{cu}.A_c + (0.75f_y-0.40f_{cu}).(A_{sc}+A_{sc+})$	**99127**	kN	*cl.3.8.4.3*
(Note for perfect axial conditions, $N_{cap} = 0.45f_{cu}.A_c + (0.95f_y-0.45f_{cu}).(A_{sc}+A_{sc+})$)			
Axial capacity utilisation = N/N_{cap}	**101%**		**NOT OK**

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Method 3A (Axial Force; Small Assumed Moments for <15% Adjacent Spans Difference in Continuou

Approximate method for allowing for moments: multiply the axial load from the floor immediately above the column being considered) by:

1.25-interior columns
1.50-edge columns
2.00-corner columns

but keep the columns to constant size for the top two storeys.

Percentage of reinforcement $(A_{sc}+A_{sc+})/A_c \times 100\%$	1.91	%	
Axial capacity, $N_{cap} = 0.35f_{cu}.A_c + (0.67f_y-0.35f_{cu}).(A_{sc}+A_{sc+})$	**87052**	kN	*cl.3.8.4.4*
Axial capacity utilisation = N/N_{cap}	**115%**		**NOT OK**

Ultimate resistance of braced stocky columns (f_{cu} = 35)

Column size & braced, clear storey height limit (mm)					Area of section ($mm^2 \times 10^3$)	p=1% (kN)	p=2% (kN)	p=3% (kN)	p=4%* (kN)
< 3530	< 4411	< 5294	< 6176	< 7059					
200 x 450	250 x 360	300 x 300			90	1369	1635	1901	2168
200 x 525	250 x 420	300 x 350			105	1597	1908	2218	2529
200 x 615	250 x 490	300 x 410	350 x 350		122.5	1863	2225	2588	2950
200 x 700	250 x 560	300 x 470	350 x 400		140	2129	2543	2958	3372
200 x 800	250 x 640	300 x 540	350 x 460	400 x 400	160	2433	2907	3380	3854
200 x 900	250 x 720	300 x 600	350 x 520	400 x 450	180	2737	3270	3803	4335
200 x1000	250 x 800	300 x 670	350 x 575	400 x 500	200	3041	3633	4225	4817
200 x1200	250 x 960	300 x 800	350 x 690	400 x 600	240	3650	4360	5070	5781

* Note : Scheme design based on 4% rebar should be avoided if possible.

The ultimate loads that can be carried by columns of different sizes and different reinforcement percentages p may be obtained from Table 5 for f_{cu} = 30N/mm² and f_y = 460N/mm².

Table 5 Ultimate loads for stocky columns

Column size* mm x mm	Cross-sectional area, mm²	p = 1% kN	p = 2% kN	p = 3% kN	p = 4% kN
300 x 300	90 000	1213	1481	1749	2016
300 x 350	105 000	1415	1728	2040	2353
350 x 350	122 500	1651	2016	2380	2745
400 x 350	140 000	1887	2304	2720	3137
400 x 400	160 000	2156	2633	3109	3585

*Provided that the smallest dimension is not less than 200mm, any shape giving an equivalent area may be used.

Method 3B (Axial Force; Small Assumed Moments; Short Column Crushing; Arup Scheme Design)

Approximate method for allowing for moments: multiply the axial load from the floor immediately above the column being considered) by:

1.25-interior columns
1.50-edge columns
2.00-corner columns

but keep the columns to constant size for the top two storeys.

Minimum column dimensions for 'stocky', braced column = clear height / 17.7

Column area where f_{cu} = 35 N/mm² and f_y = 460 N/mm² is as follows (N is axial force in Newtons):-

1% steel : Area = N/15
2% steel : Area = N/18
3% steel : Area = N/21

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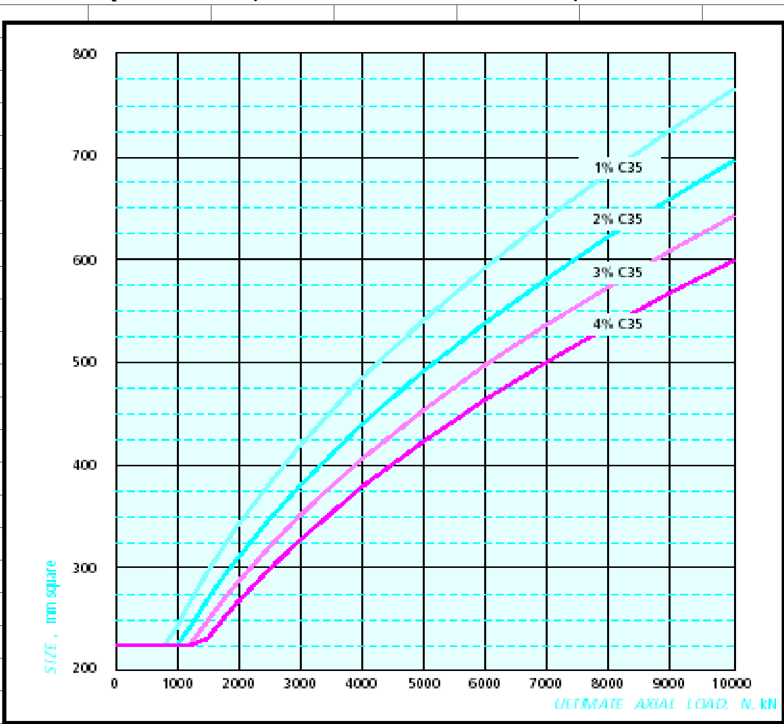
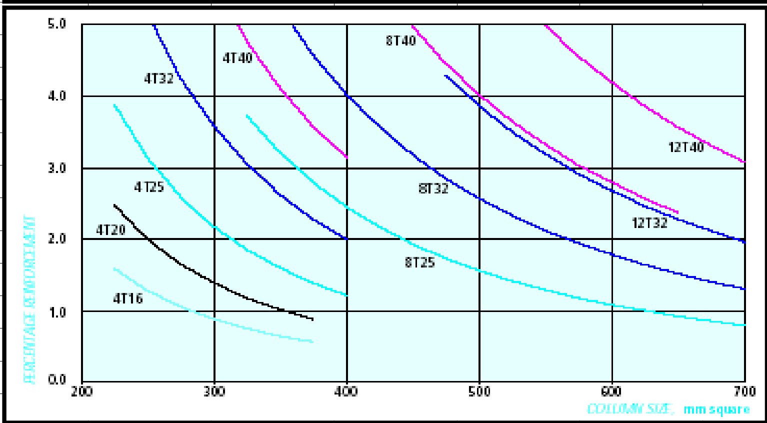
Method 3C (Axial Force; Small Assumed Moments; Short Column Crushing; Economic Concrete Scher



Internal columns



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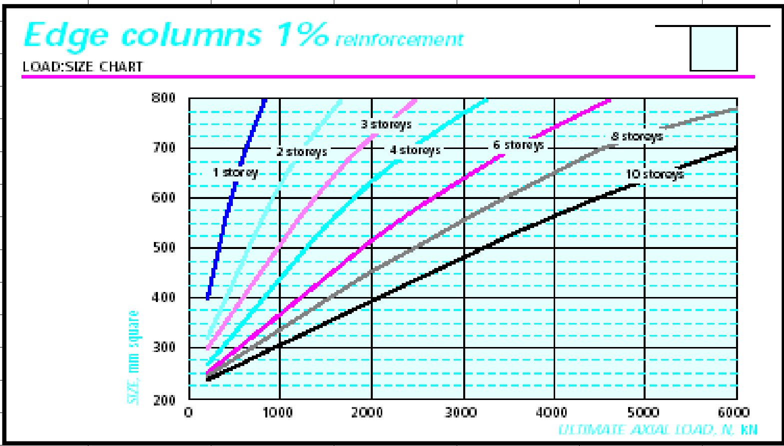

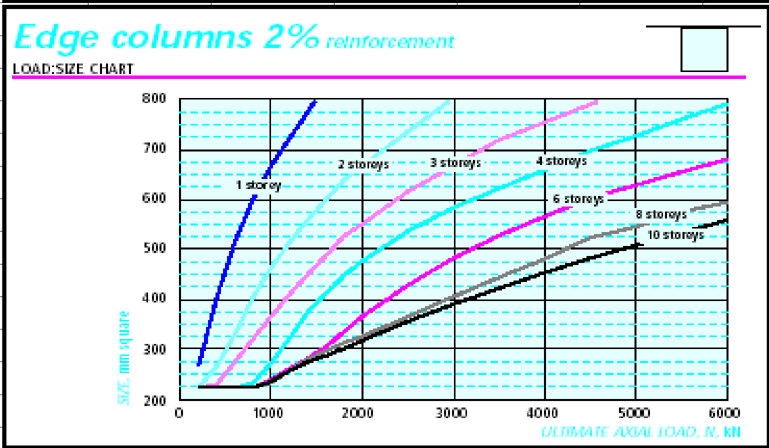

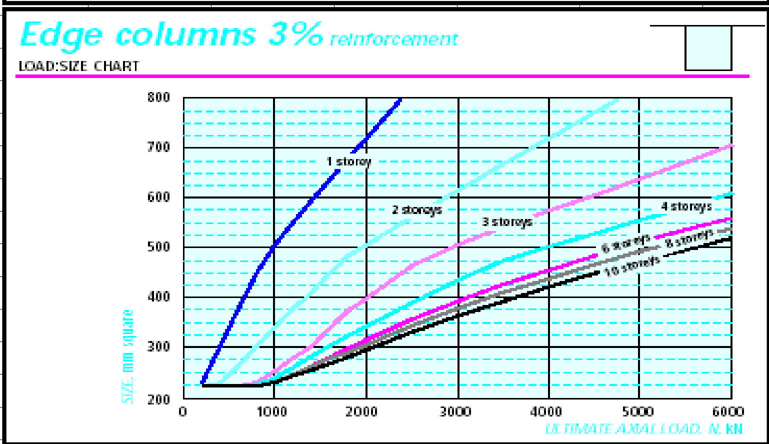

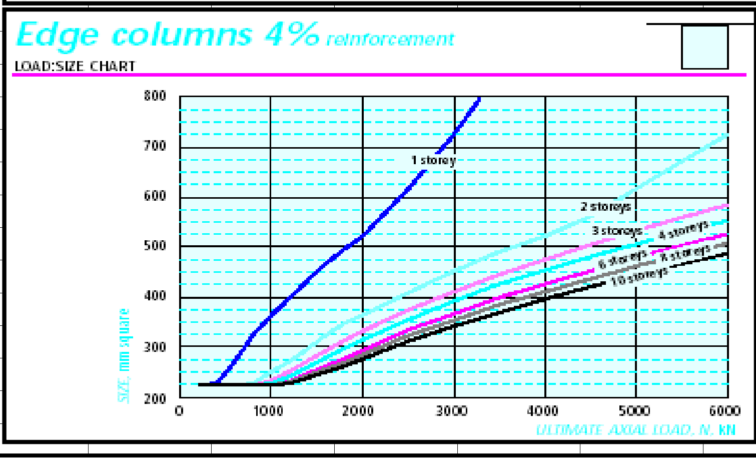

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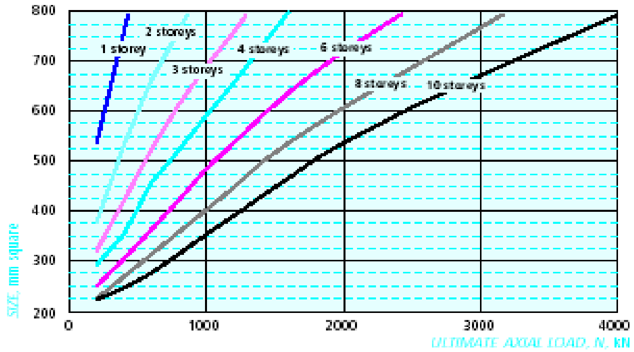
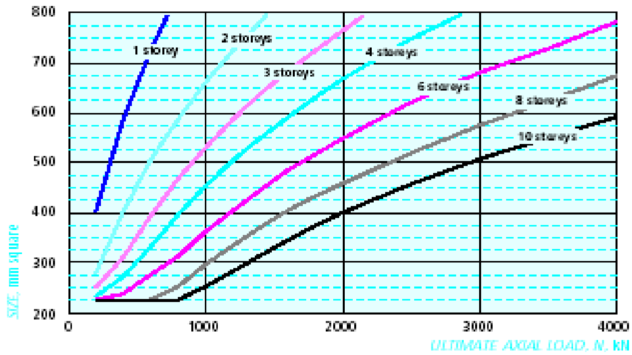
Corner columns 1% reinforcement

LOAD:SIZE CHART



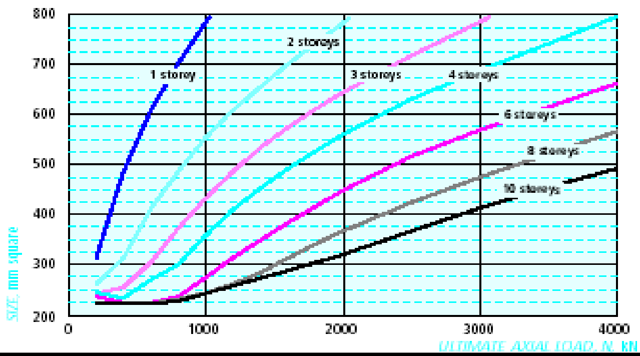
Corner columns 2% reinforcement

LOAD:SIZE CHART



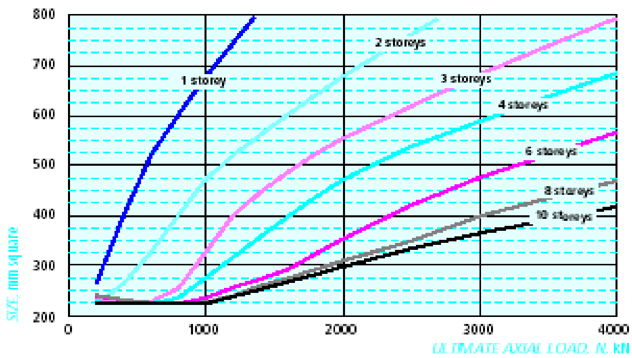
Corner columns 3% reinforcement

LOAD:SIZE CHART



Corner columns 4% reinforcement

LOAD:SIZE CHART



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Method 4 (Axial Force; Design Biaxial Moments; Short Column Crushing or Slender Column Imperfec			
(Note where relevant (h and h') or (b and b') depending on equivalent single axis of bending, for rect only)			
Depth to compression steel, h_c' = (h or b for rect, D for circ) - (h' or b')	53	mm	
Area of section, A_c	2610000	mm^2	
Ratio (h' or b')/(h or b) (rect) or (h'-h_c')/D (circ)	**0.94**		
Strength of concrete, f_{cu}	**80**	N/mm^2	
Yield strength of longitudinal steel, f_y	**460**	N/mm^2	
Rectangular ratio N/bh or circular ratio N/D^2	**38.31**	N/mm^2	
Rectangular ratio (M/bh^2 or M/hb^2) or circular ratio M/D^3	**1.13**	N/mm^2	
Perform iteration	**Design Column (Iterative)**		
Iterate depth of neutral axis until the two A_s expression equal, x	1063	mm	
Steel strain, ε_s = -ε_{cu} (h' or b' - x)/x	0.00063		
Steel strain, ε_{sc} = ε_{cu} (x-h_c')/x	0.00295		
For $f_{cu} \le 60$ N/mm², $\varepsilon_{cu} = 0.0035$			BC2 cl.2.5.3
For $f_{cu} > 60$ N/mm², $\varepsilon_{cu} = 0.0035 - (f_{cu} - 60)/50000$			cl.2.5.3
Steel design yield strength = 460/1.05 (G460) or 250/1.05 (G250)	438	N/mm^2	
Steel elastic modulus, E_s	205000	N/mm^2	
Steel stress, f_s = $E_s.\varepsilon_s$ (< design yield strength)	129	N/mm^2	
Steel stress, f_{sc} = $E_s.\varepsilon_{sc}$ (< design yield strength) - 0.45f_{cu}	402	N/mm^2	
Rectangular			
Concrete strain, ε_0 $\quad 2.4\times10^{-4}\sqrt{\frac{f_{cu}}{\gamma_m}}$	0.00175		
Factor, k_1 $\quad \frac{0.45f_{cu}}{\varepsilon_{cu}}\left(\varepsilon_{cu} - \frac{\varepsilon_0}{3}\right) = k_1$	29.2	N/mm^2	
Factor, k_2 $\quad \left[\frac{(2-\varepsilon_0/\varepsilon_{cu})^2+2}{4(3-\varepsilon_0/\varepsilon_{cu})}\right] = k_2$	0.417		
For $f_{cu} \le 60$ N/mm², $\varepsilon_{cu} = 0.0035$			BC2 cl.2.5.3
For $f_{cu} > 60$ N/mm², $\varepsilon_{cu} = 0.0035 - (f_{cu} - 60)/50000$			cl.2.5.3
A_s = [N-k_1.(b or h).x] / (f_{sc}+f_s)	18708	mm^2	**OK**
A_s = [M-k_1.(b or h).x.(0.5(h or b)-k_2.x)] / [(f_{sc}-f_s).((h' or b')-0.5(h	18691	mm^2	**OK**
$A_{sc,req}$ = MAX (2.average(A_s), 0.40%A_c) if soln; from interaction cha	37597	mm^2	
100$A_{sc,req}$/A_c	**1.44**	%	
Circular			
From interaction charts, $A_{sc,req}$	N/A	mm^2	**N/A**
100$A_{sc,req}$/A_c	**N/A**	%	
Area of longitudinal steel reinforcement required (uniaxial bending), $A_{sc,req}$	37597	mm^2	
Area of longitudinal steel reinforcement provided (uniaxial bending), A_{sc}	40212	mm^2	
Axial capacity utilisation = $A_{sc,req}$/A_{sc}	**93%**		**OK**
Convergence of interaction equations	**Converged**		

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Scheme Design

Tables 2.21 to 2.23 may be used for initial sizing. This is a summary of the data contained in *Economic concrete frame elements*[2] and should be used with the following cautions:

- Loads are ultimate loads in kN.
- Internal columns are assumed to support slabs or beams of similar spans in each orthogonal direction.
- Imposed moments on edge and corner columns have been assumed; for imposed loads greater than 5.0 kN/m^2 alternative justification is required.
- Columns are 'short' and 'braced'.

Concrete columns can be concealed within partitions by using 'blade' columns. Often a 200 x 800 mm section is used because 200 mm is a practical minimum thickness and 800 mm is four times the thickness, which classifies it as a wall. For fire resistance this reduces the cover requirements compared with a column.

Table 2.21
Initial sizing for internal square columns (mm)

Percentage of reinforcement	Ultimate axial load, kN (Class C28/35 concrete)								
	1000	1500	2000	3000	4000	5000	6000	8000	10000
1.0%	240	295	345	420	485	540	595	685	765
2.0%	225	270	310	380	440	490	540	620	695
3.0%	225	250	285	350	405	455	500	570	640
4.0%	225	230	270	330	380	425	465	535	595

Table 2.22
Initial sizing for square edge columns (mm)

	Ultimate axial load, kN (3% rebar, class C28/35 concrete)								
	400	800	1200	1600	2000	3000	4000	5000	6000
2 storeys	230	305	380	450	505				
3 storeys	225	235	280	340	400	505	575		
4 storeys	225	225	260	305	345	435	505	555	
6 storeys	225	225	250	280	315	395	455	515	560

Table 2.23
Initial sizing for square corner columns (mm)

	Ultimate axial load, kN (3% rebar, class C28/35 concrete)								
	200	400	600	800	1000	1200	1600	2000	3000
2 storeys	265	315	410	485	555	–	–	–	–
3 storeys	245	255	305	375	435	485	574	–	–
4 storeys	245	235	270	300	360	410	490	559	–
6 storeys	240	225	225	240	275	315	385	450	569