UNIVERSITI SAINS MALAYSIA

First Semester Examination
Academic Session 2008/2009

November 2008

REG 367 – Design Of Concrete Structures
(Reka Bentuk Struktur Konkrit)

Duration: 3 hours
(Masa: 3 jam)

Please check that this examination paper consists of **TWENTY TWO** pages of printed material before you begin the examination.

*Sila pastikan bahawa kertas peperiksaan ini mengandungi **DUA PULUH DUA** muka surat yang bercetak sebelum anda memulakan peperiksaan ini.*

Students are allowed to answer all questions either in English OR in Bahasa Malaysia only

Pelajar dibenarkan menjawab semua soalan dalam Bahasa Inggeris ATAU Bahasa Malaysia sahaja.

Answer **FIVE** question only.

*Jawab **LIMA** soalan sahaja.*

...2/-

1. (a) In reinforced concrete design, the slabs are designed either one way slab or two way slab depending on the span/depth ratio. How do you differentiate between one way slab and two way slab. You may give sketches to illustrate the difference in terms of load distribution and design criteria.

Dalam rekabentuk konkrit, papak lantai direka bentuk sama ada papak satu arah atau papak dua arah dan bergantung pada nisbah rentang/tebal. Bagaimanakah anda membezakan di antara 2 jenis papak lantai ini. Anda boleh gunakan lakaran untuk menerangkan perbezaan ini, khususnya agihan beban dan criteria reka bentuk.

(6 marks/*markah*)

(b) A reinforced concrete floor slab measuring 3 m x 5 m is simply supported at the four edges. The slab is assumed to be 175 mm thick with a total dead load of 5.0 kN/m^2 and imposed load of 3.0 kN/m^2. Design the reinforced concrete slab using grade 25 concrete and steel reinforcement using grade 250.

Sebuah lantai konkrit tetulang berukuran 3 m x 5 m disokong mudah oleh empat penjuru. Papak lantai ini dianggarkan setebal 175 mm dan memikul beban mati sebesar 5.0 kN/m^2 dan beban kenaan 3.0 kN/m^2. Lakukan reka bentuk papak lantai ini dengan menggunakan konkrit dari gred 25 dan tetulang keluli gred 250.

(14 marks/*markah*)

2. (a) Define or illustrate the difference between following columns:-

(i) a short column and a slender column

(ii) a braced column and an unbraced column

Takrifkan atau terangkan perbezaan antara tiang-tang berikut:-

(i) Tiang pendek dan tiang langsing

(ii) Tiang rembat dan tiang tak rembat

(6 marks/*markah*)

...3/-

(b) A short braced axially loaded reinforced concrete column of size 300 mm x 300 mm is designed to carry an ultimate axial load of 1500 kN. Find the total steel area required for the longitudinal reinforcement and the suitable size of the bars to be used. The column material is made from grade 30 concrete, the main steel reinforcement is of grade 460 and the links is made of steel of grade 250.

Sebuah tiang konkrit tetulang yang dirembat berukuran 300 mm x 300 mm telah direkabentuk untuk memikul bebanan paksian muktamad sebesar 1500 kN. Tentukan jumlah tetulang keluli utama dan saiz bar yang sesuai digunakan. Bahan tiang ini dibuat daripada konkrit gred 30, tetulang utama daripada keluli gred 460 dan pengikat tetulang ialah daripada keluli gred 250.

(14 marks/*markah*)

3. (a) Explain the difference between design moment (M_d) and ultimate moment resistance (M_u).

Terangkan perbezaan di antara momen rekabentuk (M_d) dan momen muktamad (M_u).

(5 marks/*markah*)

(b) Calculate the bending moment of the simply supported beam as shown in the following **Figure 1.0**.

(i) Load Factor method with a load factor = 1.8

(ii) Ultimate Limit State

*Kira momen rekabentuk untuk rasuk mudah seperti ditunjukkan dalam **Rajah 1.0** berikut.*

(i) Kaedah Faktor Beban dengan faktor beban = 1.8

(ii) Keadaan Had muktamad

(15 marks/*markah*)

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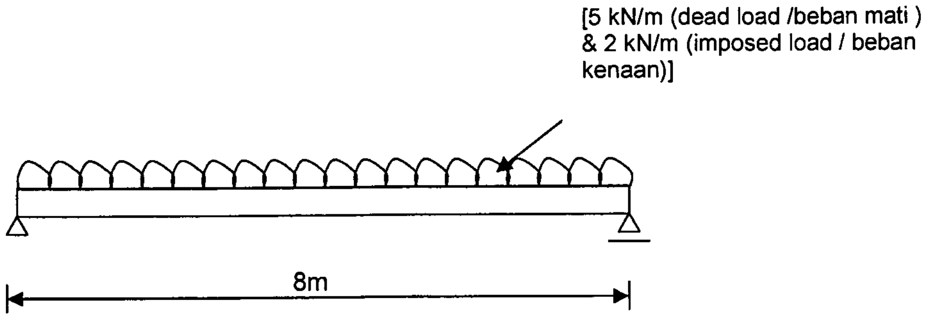

Rajah 1.0 ***(Figure 1.0)***

4. (a) What is the effect of bearing capacity of soil on the size of pad footing.

Apakah kesan keupayaan galas tanah terhadap saiz asas pad.

(5 marks/*markah*)

(b) Design a square pad footing to resist a dead load of 800 kN and imposed load of 550 kN. Assume the characteristic strength of concrete and steel is 30 N/mm^2 and 460 N/mm^2, respectively. The soil bearing capacity of the foundation is 160 kN/m^2.

Rekabentuk asas segiempat asas pad untuk menanggung beban mati 800 kN dan beban kenaan 550 kN. Anggap kekuatan ciri konkrit dan keluli adalah masing-masing 30 N/mm^2 dan 460 N/mm^2. Keupayaan galas tanah untuk asas adalah 160 kN/m^2

(15 marks/*markah*)

5. (a) Discuss and explain the characteristics of reinforced concrete. Provide sketches to support your discussion.

Bincang dan huraikan ciri-ciri konkrit bertetulang. Sertakan lakaran bagi menyokong perbincangan anda.

(10 marks/*markah*)

....5/-

(b) Describe **Five (5)** causes of failures in concrete structure.

*Huraikan **Lima (5)** faktor yang mengakibatkan berlakunya kegagalan dalam struktur konkrit.*

(5 marks/*markah*)

(c) Explain the factors influencing the durability of reinforced concrete.

Jelaskan faktor-faktor yang mempengaruhi ketahanlasakan konkrit bertetulang.

(5 marks/*markah*)

6. (a) Explain what is meant by limit state design.

Jelaskan apakah yang dimaksudkan dengan rekabentuk keadaan had.

(10 marks/*markah*)

(b) A simply supported rectangular beam of 6 meter span carries a characteristic uniformly distributed dead load (including self weight of beam) of 8 kN/m and an imposed load of 6 kN/m. The breadth of the beam is 225 mm and the effective depth is 425 mm. Find the reinforcement area required. Use grade 30 concrete and high yield steel reinforcement.

Calculate the shear reinforcement required for the beam if yield strength of shear reinforcement is 250 N/mm^2.

Sebuah rasuk disokong mudah dengan rentang 6 meter menanggung beban mati teragih seragam (termasuk berat sendiri rasuk) sebanyak 8 kN/m dan beban hidup teragih seragam sebanyak 6 kN/m. Lebar rasuk adalah 225 mm dan kedalaman berkesan adalah 425 mm. Dapatkan keluasan tetulang yang diperlukan. Gunakan konkrit gred 30 dan keluli ketegangan tinggi.

Kira tetulang ricih yang diperlukan jika kekuatan alah tetulang ricih adalah 250 N/mm^2

(10 marks/*markah*)

Table 1. Design ultimate bending moments and shear forces

	At outer support	Near middle of end span	At first interior support	At middle of interior spans	At interior supports
Moment	0	$0.09Fl$	$-0.11Fl$	$0.07FL$	$-0.08Fl$
Shear	$0.45F$	-	$0.6F$	-	$0.55F$
NOTE.	*l id the effective span;* *F is the total design ultimate load* ($1.4G_k + 1.60Q_k$).				
No redistribution of the moments calculated from this table should be made					

Table 2. Sectional areas of groups of bars (mm^2)

Bar size (mm)	Number of bars									
	1	2	3	4	5	6	7	8	9	10
6	28.3	56.6	84.9	113	142	170	198	226	255	283
8	50.3	101	151	201	252	302	352	402	453	503
10	78.5	157	236	314	393	471	550	628	707	785
12	113	226	339	452	566	679	792	905	1020	1130
16	201	402	603	804	1010	1210	1410	1610	1810	2010
20	314	628	943	1260	1570	1890	2200	2510	2830	3140
25	491	982	1470	1960	2450	2950	3440	3930	4420	4910
32	804	1610	2410	3220	4020	4830	5630	6430	7240	8040
40	1260	2510	3770	5030	6280	7540	8800	10100	11300	12600

Table 3. Perimeters and weights of bars

Bar size (mm)	6	8	10	12	16	20	25	32	40
Perimeter (mm)	18.85	25.1	31.4	37.7	50.2	62.8	78.5	100.5	125.6
Weight (kg/m)	0.222	0.395	0.616	0.888	1.579	2.466	3.854	6.313	9.864

Bar weights based on density of 7850 kg/m^3

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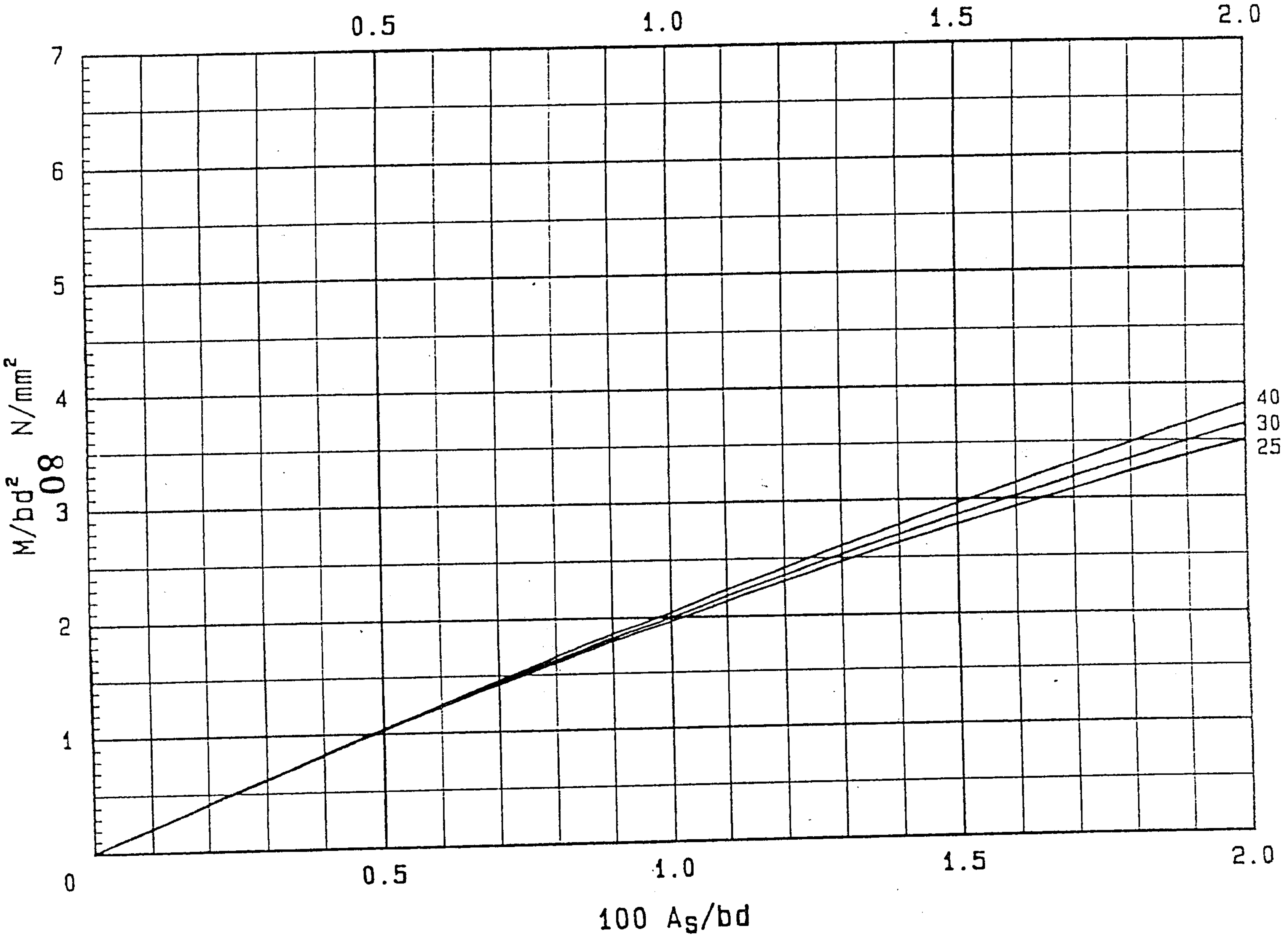
Table 4. Sectional areas per metre width for various bar spacings (mm^2)

Bar size (mm)	*Spacing of bars*								
	50	75	100	125	150	175	200	250	300
6	566	377	283	226	189	162	142	113	94
8	1010	671	503	402	335	287	252	201	168
10	1570	1050	785	628	523	449	393	314	282
12	2260	1510	1130	905	754	646	566	452	377
16	4020	2680	2010	1610	1340	1150	1010	804	670
20	6280	4190	3140	2510	2090	1800	1570	1260	1050
25	9820	6550	4910	3930	3270	2810	2450	1960	1640
32	16100	10700	8040	6430	5360	4600	4020	3220	2680
40	25100	16800	12600	10100	8380	7180	6280	5030	4190

Shear reinforcement

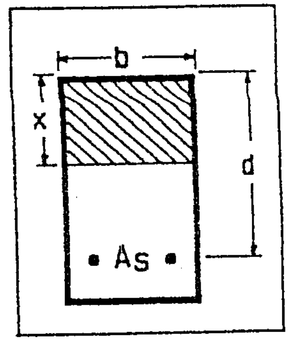
Table 5. A_{sv}/S_v for varying stirrup diameter and spacing

Stirrup diameter (mm)	*Stirrup spacing (mm)*										
	85	90	100	125	150	175	200	225	250	275	300
8	1.183	1.118	1.006	0.805	0.671	0.575	0.503	0.447	0.402	0.366	0.335
10	1.847	1.744	1.57	1.256	1.047	0.897	0.785	0.698	0.628	0.571	0.523
12	2.659	2.511	2.26	1.808	1.507	1.291	1.13	1.004	0.904	0.822	0.753
16	4.729	4.467	4.02	3.216	2.68	2.297	2.01	1.787	1.608	1.462	1.34



Singly reinforced beams

Chart No.



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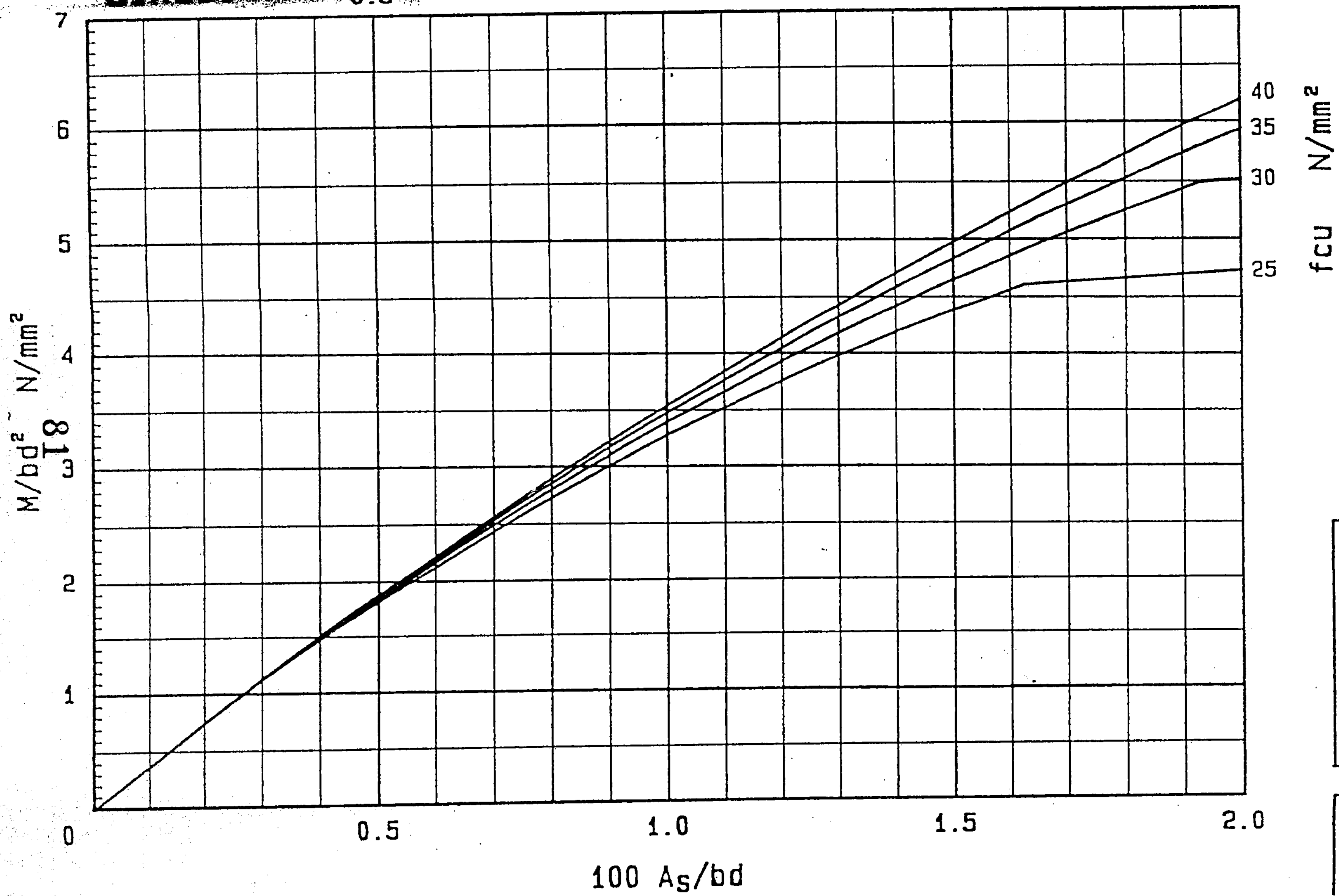

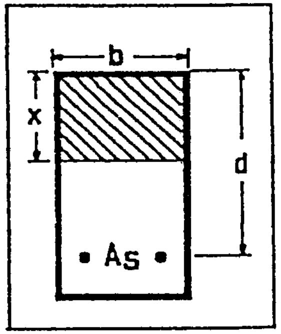

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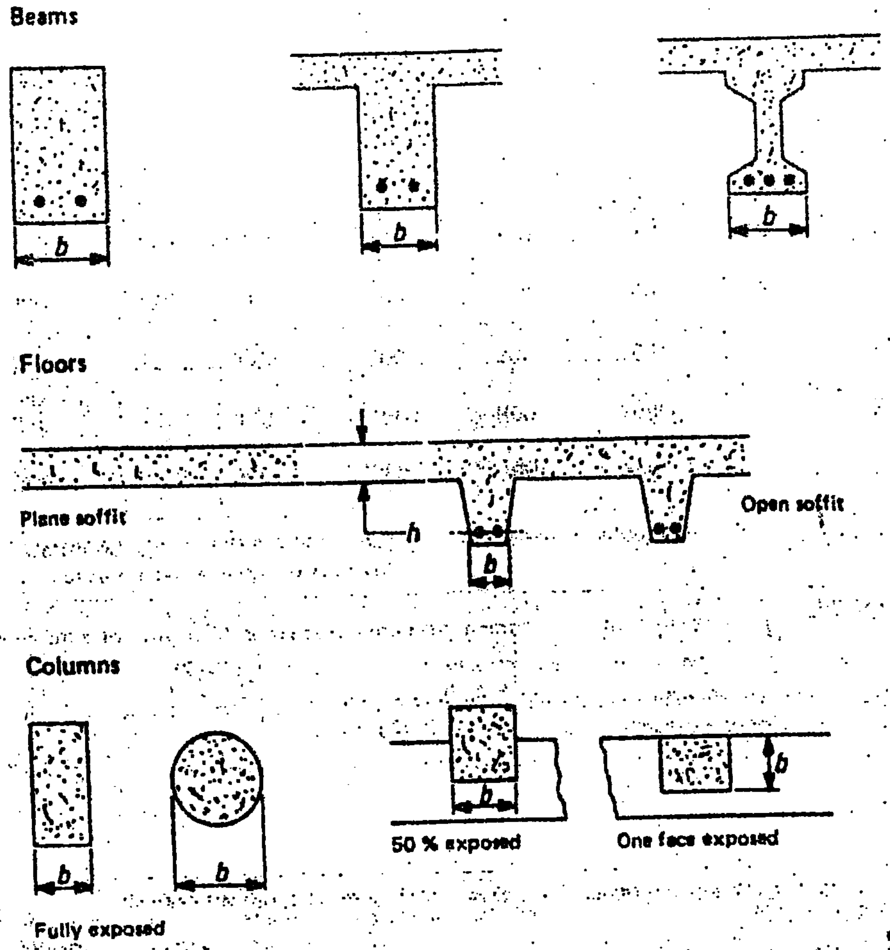
Singly reinforced beams



Fire resistance	Minimum beam width (b)	Rib width (b)	Minimum thickness of floors (h)	Column width (b)			Minimum wall thickness		
				Fully exposed	50 % exposed	One face exposed	$\rho < 0.4$ %	$0.4\% < \rho < 1\%$	$\rho > 1$ %
h	mm	mm	mm	mm	mm	mm	mm	mm	mm
0.5	200	125	75	150	125	100	150	100	75
1	200	125	95	200	160	120	150	120	75
1.5	200	125	110	250	200	140	175	140	100
2	200	125	125	300	200	160	–	160	100
3	240	150	150	400	300	200	–	200	150
4	280	175	170	450	350	240	–	240	180

NOTE 1. These minimum dimensions relate specifically to the covers given in tables 3.5 and 4.9.

NOTE 2. ρ is the area of steel relative to that of concrete.

Figure 3.2 Minimum dimensions of reinforced concrete members for fire resistance

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Table 3.4 Nominal cover to all reinforcement (including links) to meet durability requirements (see note)

Conditions of exposure (see 3.3.4)	Nominal cover				
	mm	mm	mm	mm	mm
Mild	25	20	20*	20*	20*
Moderate	–	35	30	25	20
Severe	–	–	40	30	25
Very severe	–	–	50†	40†	30
Extreme	–	–	–	60†	50
Maximum free water/cement ratio	0.65	0.60	0.55	0.50	0.45
Minimum cement content (kg/m³)	275	300	325	350	400
Lowest grade of concrete	C30	C35	C40	C45	C50

*These covers may be reduced to 15 mm provided that the nominal maximum size of aggregate does not exceed 15 mm.

†Where concrete is subject to freezing whilst wet, air-entrainment should be used (see 3.3.4.2).

NOTE 1. This table relates to normal-weight aggregate of 20 mm nominal maximum size.

NOTE 2. For concrete used in foundations to low rise construction, see 6.2.4.1).

Table 3.5 Nominal cover to all reinforcement (including links) to meet specified periods of fire resistance (see notes 1 and 2)

Fire resistance	Nominal cover						
	Beams*		Floors		Ribs		Columns*
	Simply supported	Continuous	Simply supported	Continuous	Simply supported	Continuous	
h	mm	mm	mm	mm	mm	mm	mm
0.5	20†	20†	20†	20†	20†	20†	20†
1	20†	20†	20	20	20	20†	20†
1.5	20	20†	25	20	35	20	20
2	40	30	35	25	45	35	25
3	60	40	45	35	55	45	25
4	70	50	55	45	65	55	25

*For the purposes of assessing a nominal cover for beams and columns, the cover to main bars which would have been obtained from tables 4.2 and 4.3 of BS 8110 : Part 2 : 1985 have been reduced by a notional allowance for stirrups of 10 mm to cover the range 8 mm to 12 mm (see also 3.3.6).

†These covers may be reduced to 15 mm provided that the nominal maximum size of aggregate does not exceed 15 mm (see 3.3.1.3)

NOTE 1. The nominal covers given relate specifically to the minimum member dimensions given in figure 3.2. Guidance on increased covers necessary if smaller members are used is given in section four of BS 8110 : Part 2 : 1985.

NOTE 2. Cases that lie below the bold line require attention to the additional measures necessary to reduce the risks of spalling (see section four of BS 8110 : Part 2 : 1985).

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Table 3.8 Form and area of shear reinforcement in beams

Value of v (N/mm²)	Form of shear reinforcement to be provided	Area of shear reinforcement to be provided
Less than $0.5\ v_c$ throughout the beam	See note 1	
$0.5\ v_c < v < (v_c + 0.4)$	Minimum links for whole length of beam	$A_{sv} \geqslant 0.4\ b_v s_v / 0.87\ f_{yv}$ (see note 2)
$(v_c + 0.4) < v < 0.8\sqrt{f_{cu}}$ or 5 N/mm²	Links or links combined with bent-up bars. Not more than 50 % of the shear resistance provided by the steel may be in the form of bent-up bars (see note 3)	*Where links only provided:* $A_{sv} \geqslant b_v s_v (v - v_c)/0.87\ f_{yv}$ *Where links and bent-up bars provided:* see 3.4.5.6

NOTE 1. While minimum links should be provided in all beams of structural importance, it will be satisfactory to omit them in members of minor structural importance such as lintels where the maximum design shear stress is less than half v_c.

NOTE. Minimum links provide a design shear resistance of 0.4 N/mm².

NOTE 3. See 3.4.5.5 for guidance on spacing of links and bent-up bars.

Table 3.9 Values of v_c, design concrete shear stress

$\frac{100 A_s}{b_v d}$	Effective depth (in mm)							
	125	150	175	200	225	250	300	≥ 400
	N/mm²	N/mm²	N/mm²	N/mm²	N/mm²	N/mm²	N/mm²	N/mm²
≤ 0.15	0.45	0.43	0.41	0.40	0.39	0.38	0.36	0.34
0.25	0.53	0.51	0.49	0.47	0.46	0.45	0.43	0.40
0.50	0.67	0.64	0.62	0.60	0.58	0.56	0.54	0.50
0.75	0.77	0.73	0.71	0.68	0.66	0.65	0.62	0.57
1.00	0.84	0.81	0.78	0.75	0.73	0.71	0.68	0.63
1.50	0.97	0.92	0.89	0.86	0.83	0.81	0.78	0.72
2.00	1.06	1.02	0.98	0.95	0.92	0.89	0.86	0.80
≥ 3.00	1.22	1.16	1.12	1.08	1.05	1.02	0.98	0.91

NOTE 1. Allowance has been made in these figures for a γ_m of 1.25.

NOTE 2. The values in the table are derived from the expression:

$$0.79\{100 A_s/(b_v d)\}^{1/3}(400/d)^{1/4}/\gamma_m$$

where

$\frac{100 A_s}{b_v d}$ should not be taken as greater than 3;

$\frac{400}{d}$ should not be taken as less than 1.

For characteristic concrete strengths greater than 25 N/mm², the values in table 3.9 may be multiplied by $(f_{cu}/25)^{1/3}$. The value of f_{cu} should not be taken as greater than 40.

3.4.6.4 *Long spans.* For spans exceeding 10 m, table 3.10 should be used only if it is not necessary to limit the increase in deflection after the construction of partitions and finishes. Where limitation is necessary, the values in table 3.10 should be multiplied by 10/span except for cantilevers where the design should be justified by calculation.

3.4.6.5 *Modification of span/depth ratios for tension reinforcement.* Deflection is influenced by the amount of tension reinforcement and its stress. The span/effective depth ratio should therefore be modified according to the area of reinforcement provided and its service stress at the centre of the span (or at the support in the case of a cantilever). Values of span/effective depth ratio obtained from table 3.10 should be multiplied by the appropriate factor obtained from table 3.11.

3.4.6.6 *Modification of span/depth ratios for compression reinforcement.* Compression reinforcement also influences deflection and the value of the span/effective depth ratio obtained from table 3.10 modified by the factor obtained from table 3.11 may be multiplied by a further factor obtained from table 3.12.

3.4.6.7 *Deflection due to creep and shrinkage.* Permissible span/effective depth ratios obtained from tables 3.11 to 3.15 take account of normal creep and shrinkage deflection. If it is expected that creep or shrinkage of the concrete may be particularly high (e.g. if the free shrinkage strain is expected to be greater than 0.00075 or the creep coefficient

Table 3.12 Modification factor for compression reinforcement

$\frac{100 A'_{s,prov}}{bd}$	Factor
0.00	1.00
0.15	1.05
0.25	1.08
0.35	1.10
0.50	1.14
0.75	1.20
1.0	1.25
1.5	1.33
2.0	1.40
2.5	1.45
≥ 3.0	1.50

NOTE 1. The values in this table are derived from the following equation:

$$\text{Modification factor for compression reinforcement} = 1 + \frac{100 A'_{s,prov}}{bd} \Big/ \left(3 + \frac{100 A'_{s,prov}}{bd}\right) \leq 1.5 \qquad \text{equation 9}$$

NOTE 2. The area of compression reinforcement $A'_{s,prov}$ used in this table may include all bars in the compression zone, even those not effectively tied with links.

Table 3.11 Modification factor for tension reinforcement

Service stress		M/bd^2								
		0.50	0.75	1.00	1.50	2.00	3.00	4.00	5.00	6.00
	100	2.00	2.00	2.00	1.86	1.63	1.36	1.19	1.08	1.01
	150	2.00	2.00	1.98	1.69	1.49	1.25	1.11	1.01	0.94
(f_y = 250)	156	2.00	2.00	1.96	1.66	1.47	1.24	1.10	1.00	0.94
	200	2.00	1.95	1.76	1.51	1.35	1.14	1.02	0.94	0.88
	250	1.90	1.70	1.55	1.34	1.20	1.04	0.94	0.87	0.82
(f_y = 460)	288	1.68	1.50	1.38	1.21	1.09	0.95	0.87	0.82	0.78
	300	1.60	1.44	1.33	1.16	1.06	0.93	0.85	0.80	0.76

NOTE 1. The values in the table derive from the equation:

$$\text{Modification factor} = 0.55 + \frac{(477 - f_s)}{120\left(0.9 + \frac{M}{bd^2}\right)} \leq 2.0 \qquad \text{equation 7}$$

where

M is the design ultimate moment at the centre of the span or, for a cantilever, at the support.

NOTE 2. The design service stress in the tension reinforcement in a member may be estimated from the equation:

$$f_s = \frac{5 f_y A_{s,req}}{8 A_{s,prov}} \times \frac{1}{\beta_b} \qquad \text{equation 8}$$

NOTE 3. For a continuous beam, if the percentage of redistribution is not known but the design ultimate moment at mid-span is obviously the same as or greater than the elastic ultimate moment, the stress, f_s, in this table may be taken as $5/8 f_y$.

(b) The ratio of the characteristic imposed load to the characteristic dead load does not exceed 1.25.

(c) The characteristic imposed load does not exceed 5 kN/m^2 excluding partitions.

Where analysis is carried out for the single load case of all spans loaded, the resulting support moments except those at the supports of cantilevers should be reduced by 20 %, with a consequential increase in the span moments. The resulting bending moment envelope should satisfy the provision of 3.2.2.1. No further redistribution should be carried out.

Where a span or panel is adjacent to a cantilever of length exceeding one-third of the span of the slab, the possibility should be considered of the case of slab unloaded/cantilever loaded.

3.5.2.4 *One-way spanning slabs of approximately equal span.* Where the conditions of 3.5.2.3 are met, the moments and shears in continuous one-way spanning slabs may be calculated using the coefficients given in table 3.13. Allowance has been made in these coefficients for the 20 % redistribution mentioned above.

The curtailment of reinforcement designed in accordance with table 3.14 may be carried out in accordance with the provisions of 3.12.10.

3.5.3 Solid slabs spanning in two directions at right angles: uniformly distributed loads

3.5.3.1 *General.* Subclauses 3.5.3.3 to 3.5.3.7 may be used for the design of slabs spanning in two directions at right angles and supporting uniformly distributed loads.

3.5.3.2 *Symbols.* For the purposes of 3.5.3, the following symbols apply:

l_x length of shorter side

l_y length of longer side

m_{sx} maximum design ultimate moments either over supports or at mid-span on strips of unit width and span l_x

m_{sy} maximum design ultimate moments either over supports or at mid-span on strips of unit width and span l_y

n total design ultimate load per unit area $(1.4g_k + 1.6q_k)$

N_d number of discontinuous edges $(0 \leqslant N \leqslant 4)$

v_{sx} design end shear on strips of unit width and span l_x and considered to act over the middle three-quarters of the edge

v_{sy} design end shear on strips of unit width and span l_y and considered to act over the middle three-quarters of the edge

β_{sx} sagging moment in the span, per unit width, in the direction of the shorter span, l_x, divided by nl_x^2

β_{sy} sagging moment in the span, per unit width, in the direction of the longer span, l_y, divided by nl_y^2

β_1 and β_2 hogging moments, per unit width, over the shorter edges divided by nl_x^2

β_3 and β_4 hogging moments, per unit width, over the longer edges divided by nl_y^2

α_{sx} and α_{sy} moment coefficients shown in table 3.14

β_{sx} and β_{sy} moment coefficients shown in table 3.15

β_{vx} and β_{vy} shear force coefficients shown in table 3.16

3.5.3.3 *Simply-supported slabs.* When simply-supported slabs do not have adequate provision to resist torsion at the corners, and to prevent the corners from lifting, the maximum moments per unit width are given by the following equations:

$$m_{sx} = \alpha_{sx} n l_x^2 \qquad \text{equation 10}$$

$$m_{sy} = \alpha_{sy} n l_x^2 \qquad \text{equation 11}$$

NOTE. Values for α_{sx} and α_{sy} are given in table 3.14.

The values in table 3.14 are derived from the following equations:

$$\alpha_{sx} = \frac{(l_y/l_x)^4}{8\{1 + (l_y/l_x)^4\}} \qquad \text{equation 12}$$

$$\alpha_{sy} = \frac{(l_y/l_x)^2}{8\{1 + (l_y/l_x)^4\}} \qquad \text{equation 13}$$

3.5.3.4 *Restrained slabs.* In slabs where the corners are prevented from lifting, and provision for torsion is made,

Table 3.13 Ultimate bending moment and shear forces in one-way spanning slabs

	At outer support	Near middle of end span	At first interior support	Middle of interior spans	Interior supports
Moment	0	$0.086Fl$	$-0.086Fl$	$0.063Fl$	$-0.063Fl$
Shear	$0.4F$	—	$0.6F$	—	$0.5F$

NOTE. F is the total design ultimate load $(1.4G_k + 1.6Q_k)$;
l is the effective span.

Table 3.14 Bending moment coefficients for slabs spanning in two directions at right-angles, simply-supported on four sides								
l_y/l_x	1.0	1.1	1.2	1.3	1.4	1.5	1.75	2.0
α_{sx}	0.062	0.074	0.084	0.093	0.099	0.104	0.113	0.118
α_{sy}	0.062	0.061	0.059	0.055	0.051	0.046	0.037	0.029

the maximum design moments per unit width are given by equations 14 and 15:

$$m_{sx} = \beta_{sx} n l_x^2 \quad \text{equation 14}$$

$$m_{sy} = \beta_{sy} n l_x^2 \quad \text{equation 15}$$

Where these equations are used, the conditions and rules of 3.5.3.5 should be applied.

NOTE. Values of β_{sx} and β_{sy} are given in table 3.15.

Equations 14 and 15 and the coefficients in table 3.15 may be derived from the following equations:

$$\beta_y = (24 + 2N_d + 1.5N_d^2)/1000 \quad \text{equation 16}$$

$$\gamma = \frac{2}{9}\{3 - \sqrt{(18)}\frac{l_x}{l_y}(\sqrt{\beta_y + \beta_1} + \sqrt{(\beta_y + \beta_2)})\}^2 \quad \text{equation 17}$$

$$\sqrt{\gamma} = \sqrt{(\beta_x + \beta_3)} + \sqrt{(\beta_x + \beta_4)} \quad \text{equation 18}$$

NOTE. β_1 and β_2 take values of $4/3\beta_y$ for continuous edges or zero for discontinuous edges.

β_3 and β_4 take values of $4/3\beta_x$ for continuous edges or zero for discontinuous edges.

3.5.3.5 *Restrained slabs where the corners are prevented from lifting and adequate provision is made for torsion: conditions and rules for the use of equations 14 and 15.* The conditions in which the equations may be used for continuous slabs only are as follows.

(a) The characteristic dead and imposed loads on adjacent panels are approximately the same as on the panel being considered.

(b) The span of adjacent panels in the direction perpendicular to the line of the common support is approximately the same as the span of the panel considered in that direction.

The rules to be observed when the equations are applied to restrained slabs (continuous or discontinuous) are as follows.

(1) Slabs are considered as divided in each direction into middle strips and edge strips as shown in figure 3.9, the middle strip being three-quarters of the width and each edge strip one-eighth of the width.

(2) The maximum design moments calculated as above apply only to the middle strips and no redistribution should be made.

(3) Reinforcement in the middle strips should be detailed in accordance with 3.12.10 (simplified rules for curtailment of bars)

(4) Reinforcement in an edge strip, parallel to the edge, need not exceed the minimum given in 3.12.5 (minimum areas of tension reinforcement), together with the recommendations for torsion given in (5), (6) and (7).

(5) Torsion reinforcement should be provided at any corner where the slab is simply supported on both edges meeting at that corner. It should consist of top and bottom reinforcement, each with layers of bars placed parallel to the sides of the slab and extending from the edges a minimum distance of one-fifth of the shorter span. The area of reinforcement in each of these four layers should be three-quarters of the area required for the maximum mid-span design moment in the slab.

(6) Torsion reinforcement equal to half that described in the preceding paragraph should be provided at a corner contained by edges over only one of which the slab is continuous.

(7) Torsion reinforcement need not be provided at any corner contained by edges over both of which the slab is continuous.

3.5.3.6 *Restrained slab with unequal conditions at adjacent panels.* In some cases the support moments calculated from table 3.15, for adjacent panels, may differ significantly. To adjust them the following procedures may be used.

(a) Calculate the sum of the moments at midspan and supports (neglecting signs).

(b) Treat the values from table 3.15 as fixed end moments (FEMs).

(c) Distribute the FEMs across the supports according to the relative stiffness of adjacent spans, giving new support moments.

(d) Adjust midspan moment: this should be such that when it is added to the support moments from (c) (neglecting signs) the total should equal that from (a).

If, for a given panel, the resulting support moments are now significantly greater than the value from table 3.15 the tension steel over the supports will need to be extended beyond the provisions of 3.12.10.3. The procedure should be as follows.

(e) The span moment is taken as parabolic between supports: its maximum value is as found from (d).

(f) The points of contraflexure of the new support moments (from (c)) with the span moment (from (e)) are determined.

Table 3.15 Bending moment coefficients for rectangular panels supported on four sides with provision for torsion at corners

Type of panel and moments considered	Short span coefficients, β_{sx}								Long span coefficients, β_{sy}, for all values of l_y/l_x
	Values of l_y/l_x								
	1.0	1.1	1.2	1.3	1.4	1.5	1.75	2.0	
Interior panels									
Negative moment at continuous edge	0.031	0.037	0.042	0.046	0.050	0.053	0.059	0.063	0.032
Positive moment at mid-span	0.024	0.028	0.032	0.035	0.037	0.040	0.044	0.048	0.024
One short edge discontinuous									
Negative moment at continuous edge	0.039	0.044	0.048	0.052	0.055	0.058	0.063	0.067	0.037
Positive moment at mid-span	0.029	0.033	0.036	0.039	0.041	0.043	0.047	0.050	0.028
One long edge discontinuous									
Negative moment at continuous edge	0.039	0.049	0.056	0.062	0.068	0.073	0.082	0.089	0.037
Positive moment at mid-span	0.030	0.036	0.042	0.047	0.051	0.055	0.062	0.067	0.028
Two adjacent edges discontinuous									
Negative moment at continuous edge	0.047	0.056	0.063	0.069	0.074	0.078	0.087	0.093	0.045
Positive moment at mid-span	0.036	0.042	0.047	0.051	0.055	0.059	0.065	0.070	0.034
Two short edges discontinuous									
Negative moment at continuous edge	0.046	0.050	0.054	0.057	0.060	0.062	0.067	0.070	–
Positive moment at mid-span	0.034	0.038	0.040	0.043	0.045	0.047	0.050	0.053	0.034
Two long edges discontinuous									
Negative moment at continuous edge	–	–	–	–	–	–	–	–	0.045
Positive moment at mid-span	0.034	0.046	0.056	0.065	0.072	0.078	0.091	0.100	0.034
Three edges discontinuous (one long edge continuous)									
Negative moment at continuous edge	0.057	0.065	0.071	0.076	0.081	0.084	0.092	0.098	–
Positive moment at mid-span	0.043	0.048	0.053	0.057	0.060	0.063	0.069	0.074	0.044
Three edges discontinuous (one short edge continuous)									
Negative moment at continuous edge	–	–	–	–	–	–	–	–	0.058
Positive moment at mid-span	0.042	0.054	0.063	0.071	0.078	0.084	0.096	0.105	0.044
Four edges discontinuous									
Positive moment at mid-span	0.055	0.065	0.074	0.081	0.087	0.092	0.103	0.111	0.056

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3.5.5 Shear resistance of solid slabs

3.5.5.1 *Symbols.* For the purposes of 3.5.5 the following symbols apply.

- A_{sv} area of shear links in a zone
- A_{sb} area of bent-up bars in a zone
- b breadth of slab under consideration
- d effective depth or average effective depth of a slab
- f_{yv} characteristic strength of the shear reinforcement which should not be taken as greater than 460 N/mm²
- v nominal design shear stress
- v_c design ultimate shear stress obtained from table 3.10
- V shear force due to design ultimate loads or the design ultimate value of a concentrated load
- α angle between the shear reinforcement and the plane of the slab
- s_b spacing of bent-up bars (see figure 3.4)
- s_v spacing of links

3.5.5.2 *Shear stresses.* The design shear stress, v, at any cross section should be calculated from equation 21:

$$v = \frac{V}{bd} \qquad \text{equation 21}$$

In no case should v exceed $0.8\sqrt{f_{cu}}$ or 5 N/mm², whichever is the lesser, whatever shear reinforcement is provided.

3.5.5.3 *Shear reinforcement.* Recommendations for shear reinforcement in solid slabs are given in table 3.17

3.5.6 Shear in solid slabs under concentrated loads

The provisions of 3.7.7 may be applied.

3.5.7 Deflection

Deflections may be calculated and compared with the serviceability requirements given in section three of BS 8110 : Part 2 : 1985 but, in all normal cases, it will be sufficient to restrict the span/effective depth ratio. The appropriate ratio may be obtained from table 3.10 and modified by table 3.11. Only the reinforcement at the centre of the span in the width of slab under consideration should be considered to influence deflection.

The ratio for a two-way spanning slab should be based on the shorter span and its amount of reinforcement in that direction.

3.5.8 Crack control

In general the reinforcement spacing rules given in 3.12.11 will be the best means of controlling flexural cracking in slabs, but, in certain cases, advantage may be gained by calculating crack widths (see section three of BS 8110 : Part 2 : 1985).

Table 3.17 Form and area of shear reinforcement in solid slabs

Value of v	Form of shear reinforcement to be provided	Area of shear reinforcement to be provided
N/mm² $v < v_c$	None required	None
$v_c < v < (v_c + 0.4)$	Minimum links in areas where $v > v_c$	$A_{sv} \geq 0.4\, b s_v / 0.87 f_{yv}$
$(v_c + 0.4) < v < 0.8\sqrt{f_{cu}}$ or 5 N/mm²	Links and/or bent-up bars in any combination (but the spacing between links or bent-up bars need not be less than d)	*Where links only provided:* $A_{sv} \geq b s_v (v - v_c)/0.87 f_{yv}$ . . . *Where bent-up bars only provided:* $A_{sb} \geq b s_b (v - v_c)/(0.87 f_{yv} (\cos\alpha + \sin\alpha \cot\beta))$ (see 3.4.5.7)

NOTE 1. It is difficult to bend and fix shear reinforcement so that its effectiveness can be assured in slabs less than 200 mm deep. It is therefore not advisable to use shear reinforcement in such slabs.

NOTE 2. The enhancement in design shear strength close to supports described in 3.4.5.9 and 3.4.5.10 may also be applied to solid slabs.

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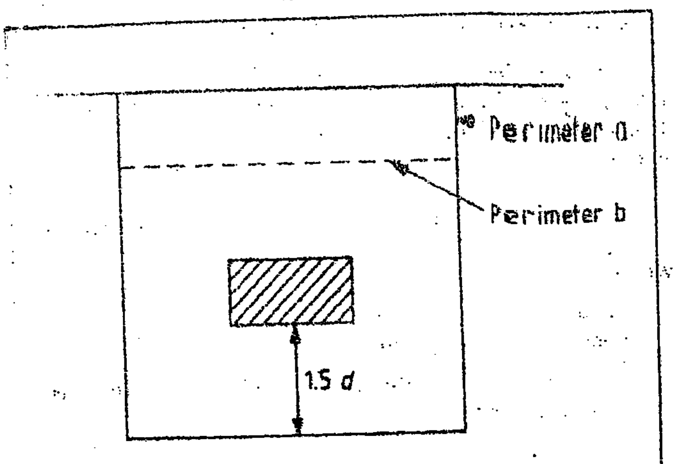

Figure 3.19 Shear perimeters with loads close to free edge

3.7.8 Deflection of panels

For slabs with drops of gross width in both directions at least equal to one-third of the respective spans, the provisions of 3.4.6 can be applied directly. Otherwise the span/effective depth ratios obtained from 3.4.6 should be multiplied by 0.9. The check should be carried out for the more critical direction.

3.7.9 Crack control in panels

In general the reinforcement spacing rules given in 3.12.11 will be the best means of controlling flexural cracking in panels but, in certain cases, advantage may be gained by calculating crack widths (see section three of BS 8110 : Part 2 : 1985) and comparing them with the required values.

3.7.10 Design of columns in flat slab construction

Columns should be designed in accordance with the provisions of 3.8.

3.8 Columns

3.8.1 General

NOTE. The provisions of this clause relate to columns whose greater overall cross-sectional dimension does not exceed four times its smaller dimension. While the provisions relate primarily to rectangular cross sections, the principles involved may be applied to other shapes where appropriate.

3.8.1.1 *Symbols.* For the purposes of 3.8 the following symbols apply.

- A_c net cross-sectional area of concrete in a column
- A_{sc}* area of vertical reinforcement
- a_u deflection at ULS for each column calculated from equation 32
- a_{uav} average deflection at ULS applied to all columns at a given level
- b width of a column (dimension of cross section perpendicular to h)
- h depth of the cross section measured in the plane under consideration
- l_e effective height of a column in the plane of bending considered
- l_{ex} effective height in respect of the major axis
- l_{ey} effective height in respect of the minor axis
- l_o clear height between end restraints
- l_c height of a column measured between centres of restraints
- M_1 smaller initial end moment due to design ultimate loads
- M_2 larger initial end moment due to design ultimate loads
- M_i initial design ultimate moment in a column before allowance for additional design moments arising out of slenderness
- M_x design ultimate moment about the x axis
- M_x' effective uniaxial design ultimate moment about the x axis
- M_y design ultimate moment about the y axis
- M_y' effective uniaxial design ultimate moment about the y axis
- M_{add} additional design ultimate moment induced by deflection of column
- N design ultimate axial load on a column
- N_{bal} design axial load capacity of a balanced section; for symmetrically-reinforced rectangular sections, it may be taken as $0.25 f_{cu} bd$
- N_{uz} design ultimate capacity of a section when subjected to axial load only
- n number of columns resisting sidesways at a given level or storey

3.8.1.2 *Size of columns.* The size of a column and the position of the reinforcement in it may be affected by the requirements for durability and fire resistance, and these should be considered before the design is commenced.

3.8.1.3 *Short and slender columns.* A column may be considered as short when both the ratios l_{ex}/h and l_{ey}/b are less than 15 (braced) and 10 (unbraced). It should otherwise be considered as slender.

3.8.1.4 *Plain concrete columns.* If a column has a large enough section to resist the ultimate loads without the addition of reinforcement, then it may be designed similarly to a plain concrete wall (see 1.2.4).

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.

* A_{sc} the suffix here denotes main reinforcement in columns. It does not necessarily imply that the reinforcement will be in compression. ...19/

3.8.1.6 *Effective height of a column*

3.8.1.6.1 *General.* The effective height, l_e, of a column in a given plane may be obtained from the following equation:

$$l_e = \beta l_o \qquad \text{equation 30}$$

Values of β are given in tables 3.21 and 3.22 for braced and unbraced columns respectively as a function of the end conditions of the column. Formulae that may be used to obtain a more rigorous assessment of the effective length, if desired, are given in 2.5 of BS 8110 : Part 2 : 1985.
It should be noted that the effective height of a column in the two plan directions may be different.

In tables 3.21 and 3.22 the end conditions are defined in terms of a scale from 1 to 4. Increase in this scale corresponds to a decrease in end fixity. An appropriate value can be assessed from 3.8.1.6.2.

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).

Table 3.21 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 3.22 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	–	–

3.8.1.7 *Slenderness limits for columns.* Generally, the clear distance, l_o, between end restraints should not exceed sixty times the minimum thickness of a column.

3.8.1.8 *Slenderness of unbraced columns.* If, in any given plane, one end of an unbraced column is unrestrained (e.g. a cantilever column), its clear height, l_o, should not exceed:

$$l_o = \frac{100b'^2}{h'} \leqslant 60b' \qquad \text{equation 3}$$

where

h' and b' are respectively the larger and smaller dimensions of the column.

The considerations of deflection (see 3.8.5) may introduce further limitations.

3.8.2 Moments and forces in columns

3.8.2.1 *Columns in monolithic frames designed to resist lateral forces.* In such cases the moments, shear forces and axial forces should be determined in accordance with 3.2.1 (see also 3.8.2.2).

3.8.2.2 *Additional moments induced by deflection at ULS.* In slender columns additional moments induced by deflection at ULS should also be considered. An allowance for them is made in the design requirements for slender columns (see 3.8.3). The bases or other members connected to the ends of such columns should also be designed to resist these additional moments at ULS if the average value of l_e/h for all columns at a particular level is greater than 20. Subclause 3.8.3.9 gives guidance in the design for these moments.

3.8.2.3 *Columns in column-and-beam construction, or in monolithic braced structural frames.* The axial force in a column may be calculated on the assumption that beams and slabs transmitting force into it are simply supported. When a column is axially loaded or the axial force dominates, as in the case of columns supporting symmetrical arrangements of beams, only the design ultimate axial force need be considered in design apart from a nominal allowance for eccentricity, equal to that recommended in 3.8.2.4.

3.8.2.4 *Minimum eccentricity.* At no section in a column should the design moment be taken as less than that produced by considering the design ultimate axial load as acting at a minimum eccentricity, e_{min}, equal to 0.05 times the overall dimension of the column in the plane of bending considered but not more than 20 mm. Where biaxial bending is considered, it is only necessary to ensure that the eccentricity exceeds the minimum about one axis at a time.

3.8.3 Deflection induced moments in solid slender columns

3.8.3.1 *Design.* In general, a cross section may be designed by the methods given for a short column (see 3.8.4) but in the design, account has to be taken of the additional moment induced in the column by its deflection.

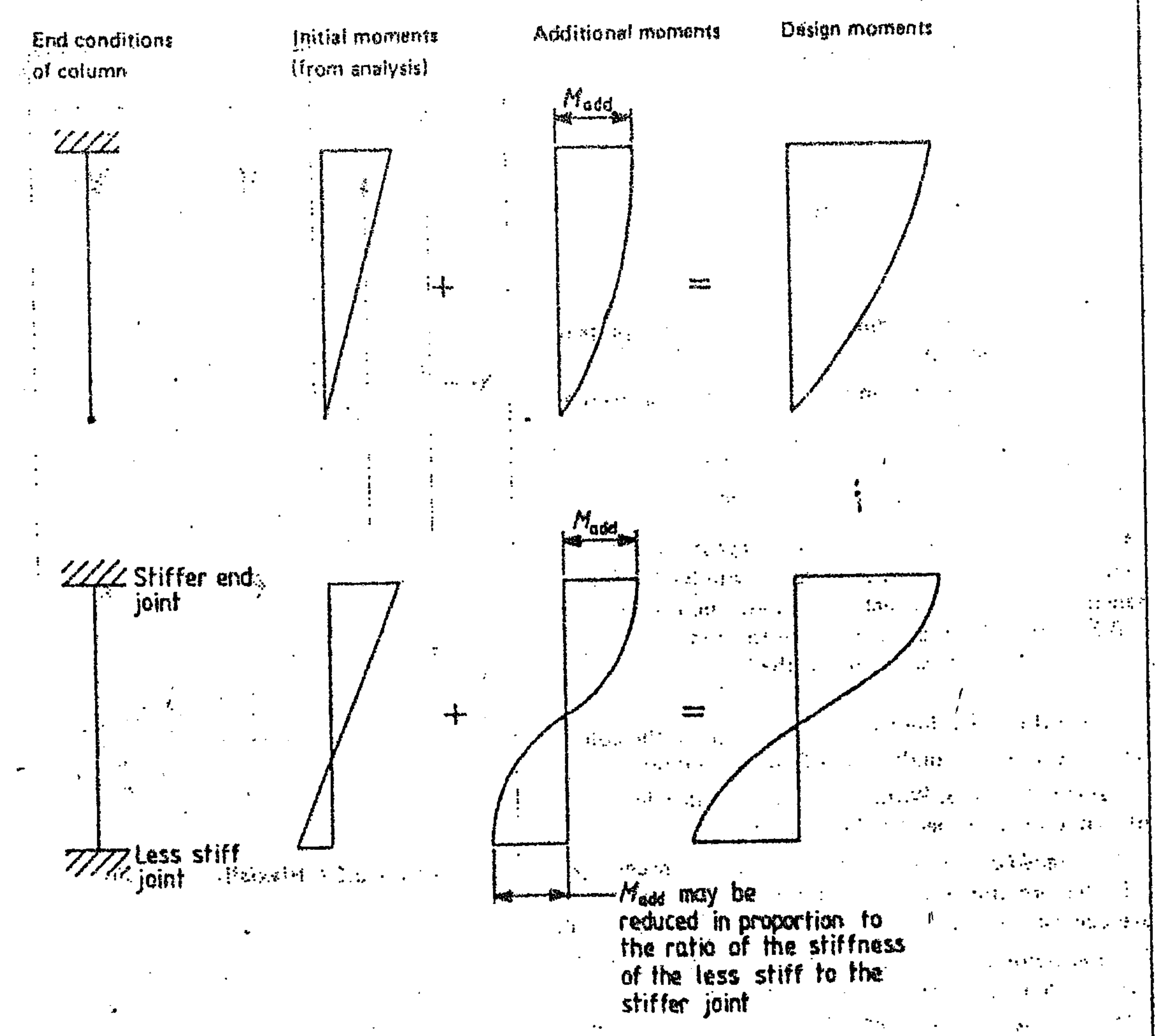

Figure 3.21 Unbraced slender columns

3.8.4 Design of column section for ULS

3.8.4.1 *Analysis of sections.* In the analysis of a column cross section to determine its design ultimate resistance to moment and axial force, the same assumptions should be made as when analysing a beam (see 3.4.4.1).

3.8.4.2 *Design charts for symmetrically-reinforced columns.* Design charts for symmetrically-reinforced columns are given in BS 8110 : Part 3. They are based on figures 2.1 and 2.2 of this code and the assumptions of 3.4.4.1.

3.8.4.3 *Nominal eccentricity of short columns resisting moments and axial forces.* Short columns usually need only to be designed for the maximum design moment about the one critical axis.

Where, due to the nature of the structure, a column cannot be subjected to significant moments, it may be designed so that the design ultimate axial load does not exceed the value of N given by:

$$N = 0.4 f_{cu} A_c + 0.75 A_{sc} f_y \qquad \text{equation 38}$$

NOTE. This includes an allowance for γ_m.

3.8.4.4 *Short braced columns supporting an approximately symmetrical arrangement of beams.* The design ultimate axial load for a short column of this type may be calculated using the following equation:

$$N = 0.35 f_{cu} A_c + 0.67 A_{sc} f_y \qquad \text{equation 39}$$

where

(a) the beams are designed for uniformly distributed imposed loads, and

(b) the beam spans do not differ by more than 15 % of the longer.

NOTE. This includes an allowance for γ_m.

3.8.4.5 *Biaxial bending.* When it is necessary to consider biaxial bending and in the absence of more rigorous calculations in accordance with 3.4.4.1, symmetrically-reinforced rectangular sections may be designed to withstand an increased moment about one axis given by the following equations:

(a) for $M_x/h' \geqslant M_y/b'$, $M_x' = M_x + \beta \frac{h'}{b'} M_y$ equation 40

(b) for $M_x/h < M_y/b'$, $M_y' = M_y + \beta \frac{b'}{h'} M_x$ equation 41

where

h' is the overall section dimension in a direction perpendicular to the x axis;

b' is the overall section dimension perpendicular to the y axis;

β is a coefficient obtained from table 3.24 below.

NOTE. See figure 3.22 for further clarification of b' and h'.

3.8.4.6 *Shear in columns.* The design shear strength of columns may be checked in accordance with 3.4.5.13. For rectangular sections, no check is required where M/N is less than $0.75h$ provided that the shear stress does not exceed $0.8\sqrt{f_{cu}}$ or 5 N/mm², whichever is the lesser.

3.8.5 Deflection of columns

No check is necessary under the following conditions.

(a) *Braced columns.* Within the recommended limits of slenderness no specific check is necessary.

(b) *Unbraced columns.* No check is normally necessary if in the direction and at the level considered the average value of l_e/h for all columns is not more than 30.

(c) *Single-storey construction.* Where no finishes susceptible to damage as a result of deflection are present, an unbraced column within the recommended limits of slenderness (see 3.8.1.8) may be considered to be acceptable.

If checks are needed, guidance on appropriate limits is given in section three of BS 8110 : Part 2 : 1985.

3.8.6 Crack control in columns

Cracks due to bending in a column designed for design ultimate axial load greater than $0.2f_{cu}A_c$ are unlikely to occur and therefore no check is required. A more lightly-loaded column subject to bending should be considered as a beam for the purpose of crack control.

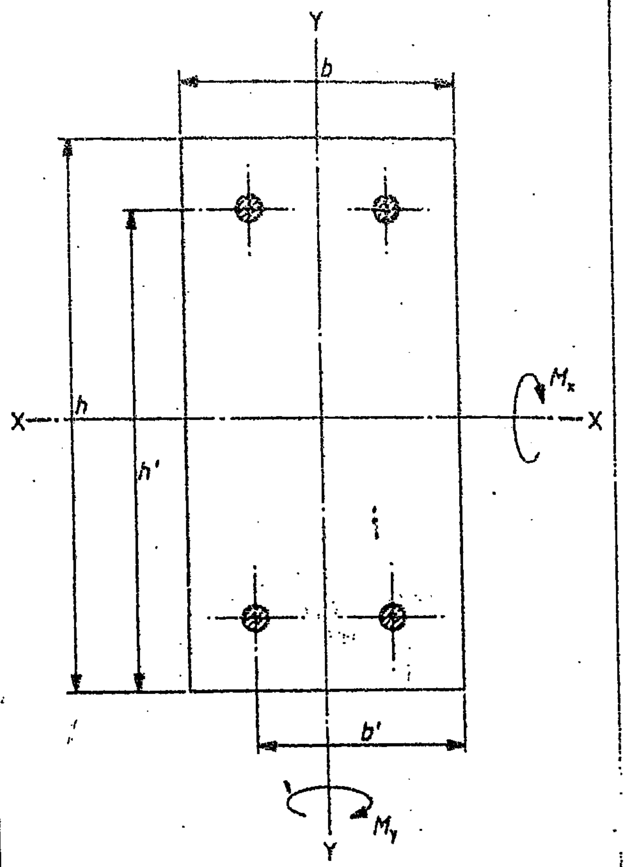

Figure 3.22 Biaxially bent column

3.9 Walls

NOTE. See 1.2.4 for definitions specific to walls.

3.9.1 Symbols

For the purposes of 3.9 the following symbols apply.

A_c gross area of concrete at a cross section

A_{sc}* area of compression reinforcement, per unit length of wall

e_a additional eccentricity due to deflections

Table 3.24 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

*The 'c' in the suffix denotes total vertical reinforcement and is justified as an extension of its use in 3.8.

3.9.4.24.2 *Shear walls.* The deflection of plain concrete shear walls should be within acceptable limits if the total height does not exceed ten times the length of the wall.

3.10 Staircases

3.10.1 General

NOTE. For the purposes of this clause, a staircase may be taken to include a section of landing spanning in the same direction and continuous with the stair flight.

3.10.1.1 *Loading.* Staircases should be designed to support the design ultimate loads according to the load combinations in 3.2.1.2.2.

3.10.1.2 *Distribution of loading.* In general, the design ultimate load should be assumed to be uniformly distributed over the plan area of a staircase. When, however, staircases surrounding open wells include two spans that intersect at right angles, the load on the areas common to both spans may be assumed to be divided equally between the two spans.

When staircases or landings that span in the direction of the flight are built at least 110 mm into walls along part or all of their length, a 150 mm strip adjacent to the wall may be deducted from the loaded area.

3.10.1.3 *Effective span of monolithic staircases without stringer beams.* When the staircase is built monolithically at its ends into structural members spanning at right angles to its span, the effective span should be as given in equation 47:

$$\text{effective span} = l_a + 0.5\,(l_{b,1} + l_{b,2}) \qquad \text{equation 47}$$

where

l_a is the clear horizontal distance between the supporting members;

$l_{b,1}$ is the breadth of the supporting member at one end *or* 1.8 m, whichever is the smaller;

$l_{b,2}$ is the breadth of the supporting member at the other end *or* 1.8 m, whichever is the smaller.

3.10.1.4 *Effective span of simply-supported staircases without stringer beams.* The effective span of simply-supported staircases without stringer beams should be taken as the horizontal distance between the centre-lines of the supports or the clear distance between the faces of supports plus the effective depth, whichever is the lesser.

3.10.1.5 *Depth of section.* The depth of the section should be taken as the minimum thickness perpendicular to the soffit of the staircase.

3.10.2 Design of staircases

3.10.2.1 *Strength, deflection and crack control.* The recommendations for beams and slabs given in 3.4 and 3.5 apply except for the span/depth ratio of a staircase without stringer beams where 3.10.2.2 applies.

3.10.2.2 *Permissible span/effective depth ratio for staircases without stringer beams.* Provided the stair flight occupies at least 60 % of the span, the ratio calculated in accordance with 3.4.6.3 may be increased by 15 %.

3.11 Bases

3.11.1 Symbols

For the purposes of 3.11 the following symbols apply.

- A_s total cross-sectional area of reinforcement parallel to the shorter side of a slab
- a_v distance from the face of a column to the critical shear section
- c column width
- c_x horizontal dimension of a column, parallel to l_x
- c_y horizontal dimension of a column, parallel to l_y
- d effective depth of a pad footing or pile cap
- h thickness of pad footing or pile cap
- l_c half the spacing between column centres (if more than one) or the distance to the edge of the pad, whichever is the greater)
- l_x length of the longer side of a base
- l_y length of the shorter side of a base
- v design shear stress at a section
- v_c design concrete shear stress (see table 3.9)
- ϕ diameter of a circular pile or of a circle inscribed in the plan form of a pile of other shape

3.11.2 Assumptions in the design of pad footings and pile caps

3.11.2.1 *General.* Except where the reactions to the applied loads and moments are derived by more accurate methods, e.g., an elastic analysis of a pile group or the application of established principles of soil mechanics, the following assumptions should be made.

(a) When a base or a pile cap is axially loaded, the reactions to design ultimate loads may be assumed to be uniformly distributed (i.e. load per unit area or per pile).

(b) When a base or a pile cap is eccentrically loaded, the reactions may be assumed to vary linearly across the base or across the pile system.

3.11.2.2 *Critical section in design of an isolated pad footing.* The critical section in design of an isolated pad footing may be taken as that at the face of the column or wall supported.

3.11.2.3 *Pockets for precast members.* Account should be taken of pockets for precast members in calculating section resistances, unless grouted up with a cement mortar not weaker than the concrete in the base.

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