UNIVERSITI TUN HUSSEIN ONN MALAYSIA

FINAL EXAMINATION
SEMESTER 1
SESSION 2015/2016

COURSE NAME	:	STRUCTURAL DESIGN
COURSE CODE	:	DAC 31903
PROGRAMME	:	3 DAA
EXAMINATION DATE	:	DECEMBER 2015/JANUARY 2016
DURATION	:	3 HOURS
INSTRUCTION	:	ANSWER **FOUR** QUESTIONS FROM SIX QUESTIONS GIVEN

THIS QUESTION PAPER CONSISTS OF **TWENTY ONE(21)** PAGES

Q1 (a) (i) Give **three (3)** functions of concrete cover.

(3 marks)

(ii) Sketch the reinforced concrete stress-block diagram based on EC2 at ultimate limit state and write the salient values.

(5 marks)

(b) **Figure Q1(b)** shows a floor plan of a double storey building. Given the following data:

Thickness of concrete slab	= 125 mm
Imposed load on floor	= 3.0 kN/m^2
Weight of concrete	= 25 kN/m^3
f_{yk}	= 500 N/mm^2
f_{ck}	= 25 N/mm^2
Weight of brick wall on the beam	= 2.0 kN/m
Floor finishes	= 1.5 kN/m^2
Cover	= 25 mm

Assuming the secondary beam 2/B-C is simply supported,

(i) calculate the design load on the beam.

(5 marks)

(ii) calculate the design moment.

(3 marks)

(iii) design the main reinforcement for the beam.

(5 marks)

(iv) check the deflection of the beam.

(4 marks)

Q2 (a) (i) Define braced column?

(2 marks)

(ii) Classify the column using the following information:

Types: Braced	
Column size	= 300 mm(b) x 400 mm (h)
Concrete characteristic strength	= C30/37
Design axial load on column	= 1200 kN
Effective height of column	= 2230 mm

(6 marks)

(b) **Figure Q2** shows a cross-section of a short braced column of size 300 x 400 mm. The effective height of the column is 3.5 m. The column is subjected to biaxial moments of M_{z-z} = 90 kNm and M_{y-y} = 70 kNm and an axial design load of 1500 kN. Given the following data:

f_{ck}	=	30 N/mm^2
f_{yk}	=	500 N/mm^2
Concrete cover	=	35 mm
Assume diameter of main bars	=	16 mm
Assume link diameter	=	8 mm

(i) Design the main reinforcement and link for the column.
(9 marks)

(ii) Sketch the detailing of the column.
(3 marks)

(iii) If the same column size is used but the biaxial bending moment is ignored, calculate the ultimate axial load that can be supported by the column. [Use the same main reinforcement as calculated in (i) and assume the column is subjected to a minimum eccentric moment]
(5 marks)

Q3 (a) Give **two(2)** differences between a one-way slab and a two-way slab.
(4 marks)

(b) **Figure Q3** shows a cast in-situ slab panel inside a building which is simply supported on steel beams on four of its sides. The size of the slab is 5 m x 6 m and the thickness is 175 mm. Given the following data:

Characteristic dead load of slab (including finishing and service)	= 6.20 kN/m^2
Characteristic imposed load	= 2.5 kN/m^2
Concrete grade	= 30 N/mm^2
Steel grade	= 500 N/mm^2
Cover	= 30 mm
Assume main reinforcement	= 12 mm

(i) Classify the type of slab.
(2 marks)

(ii) Design the main reinforcements for the slab.
(8 marks)

(iii) Check the deflection of the slab.
(4 marks)

(iv) Sketch the detailing of the slab.
(4 marks)

(v) Give one reason why in the above case a two-way slab is better than a one-way slab.
(3 marks)

Q4 (a) Define restrained beam? Sketch an example of a restrained beam.
(5 marks)

(b) **Figure Q4** shows a 8 m length simply supported steel beam of size 406 x 178 x 60 UB grade 275. The beam carries characteristic imposed point load of 100 kN at the locations shown.

Section properties 406x178x60UB (p_y=p_{yw}=275 N/mm^2):

D= 406.4 mm	b/T = 6.95	Z_x =1060 cm^3
B=177.9 mm	d/T = 45.6	Z_y =135 cm^3
t= 7.9 mm	S_x=1200cm^3	A=76.5 cm^2
T=12.3 mm	S_y=209 cm^3	r = 10.2 mm
d= 360.4 mm		

If the beam is fully restrained,

(i) Classify the beam section.
(3 marks)

(ii) Check the shear capacity of the beam.
(4 marks)

(iii) Check the beam moment capacity.
(4 marks)

(iv) Check the bearing and buckling capacity of the web at support.
(6 marks)

(v) Evaluate the type of failure that can occur on the beam if the beam is not restrained.
(3 marks)

Q5 (a) Name and sketch **two(2)** types of commonly used roof truss.

(4 marks)

(b) **Figure Q5** shows a truss used in a building. The truss is spaced at 6 m centre. Sag rods are installed at the centre of the purlin between the trusses. The size of purlin is 125 x 75 x 10L with the following properties:

Size(AxB) = 100x75 mm	$Z_x = 36.5\ cm^3$	$A_g = 19.1\ cm^2$
t= 10 mm	$Z_y = 13.3\ cm^3$	

Given the following data,

Weight of roofing sheet, insulation and purlins. (on slope)	=	$0.35\ kN/m^2$
Self weight of truss(on slope)	=	$0.2\ kN/m^2$
Imposed load (on plan)	=	$0.5\ kN/m^2$

(i) Check whether the purlin size chosen is suitable.

(6 marks)

(ii) Calculate the load that is transferred to truss nodes.

(4 marks)

(iii) Calculate the tensile force in member AB.

(4 marks)

(iv) Check the suitability of an angle of size 100x65x7L to be used for member AB if both ends are welded. (The longer leg is the connected element)

Given:

Properties of 100 x 65 x 7L:
Size(AxB) = 100 x 65 mm $A_g = 11.2\ cm^2$,
t = 7 mm

(7 marks)

Q6 (a) Briefly describe how fibre moisture content influence the strength of timber.

(5 marks)

(b) **Figure Q6** shows timber floor joist of a two storey using common dry grade timber of strength group SG3. The joists are spaced 500 mm centres and the effective span of floor is 5 m.

Properties of SG3 timber(common/dry) :

Bending parallel to grain = 12.6 N/mm^2
Modulus of elasticity = 14300 N/mm^2(mean)
10300 N/mm^2(minimum)
K_1 – duration of loading = 1.0 (long term)
K_2 – load sharing = 1.1 (more than 4 members)
K_3 – depth factor = not applicable (less than 300 mm)

Based on the following data,

Floor dead weight	= 0.2 kN/m^2
Ceiling weight	= 0.1 kN/m^2
Joist self weight	= 0.15 kN/m^2
Imposed load	= 1.5 kN/m^2

(i) Calculate the load and the maximum bending moment on the joist. (5 marks)

(ii) Check whether a joist of size 100 x 150 mm (Z_{xx} = 189 x 10^3 mm^3, I_{xx} = 20.6 x 10^{6}mm^4, A = 12600 mm^2) is suitable. (8 marks)

(iii) Check the joist deflection. (7 marks)

Given : $\delta_t = 5Wl^3/384E_{min}I_{xx}$
Permissible deflection = 0.003 x span
W = Total load(kN)

END OF QUESTIONS

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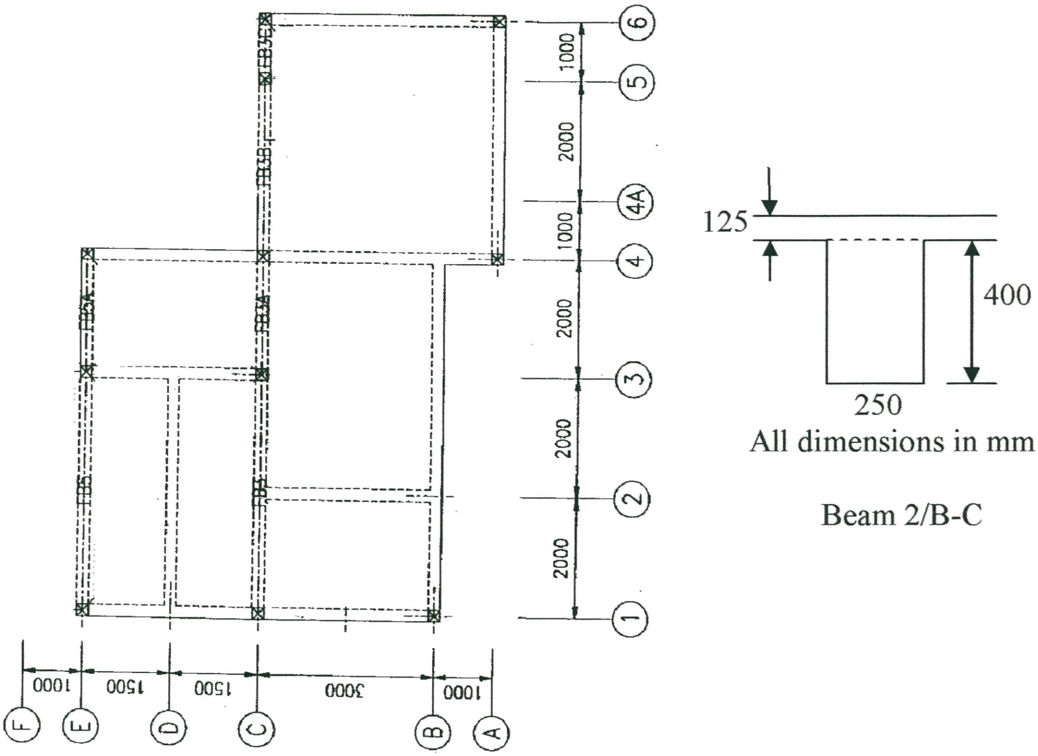

Figure Q1(b)

Estimated trapezoidal load to uniformly distributed load :

$$w = \frac{n.L_x}{6}\left[3 - \left(\frac{L_x}{L_y}\right)^2\right]$$

Estimated triangular load to uniformly distributed load:

$$w = \frac{n.L_x}{3}$$

$n = kN/m^2$

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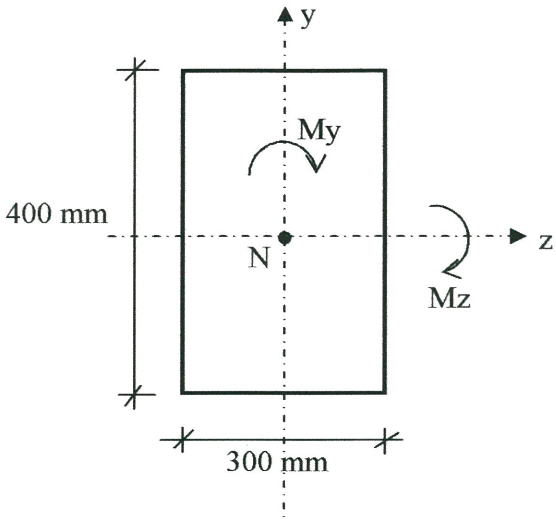

Figure Q2

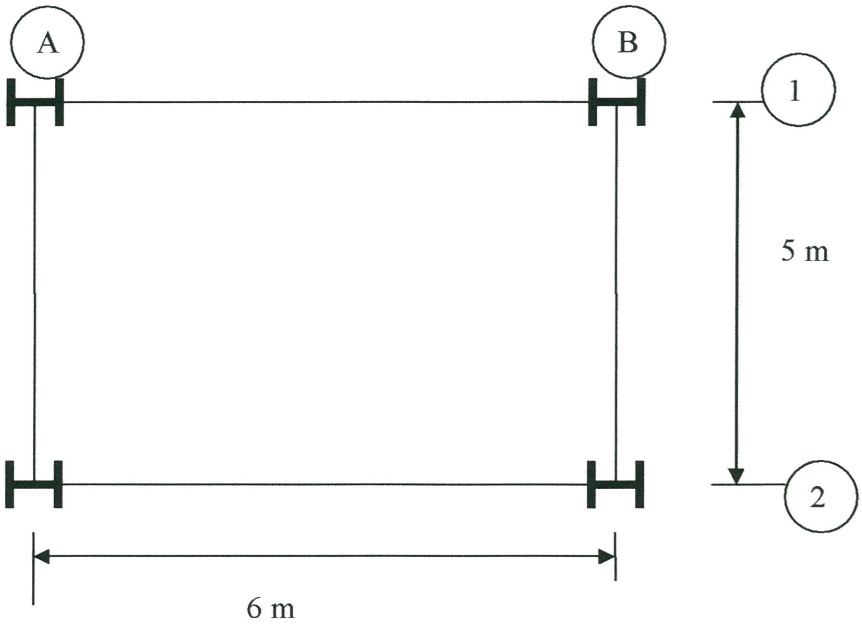

Figure Q3

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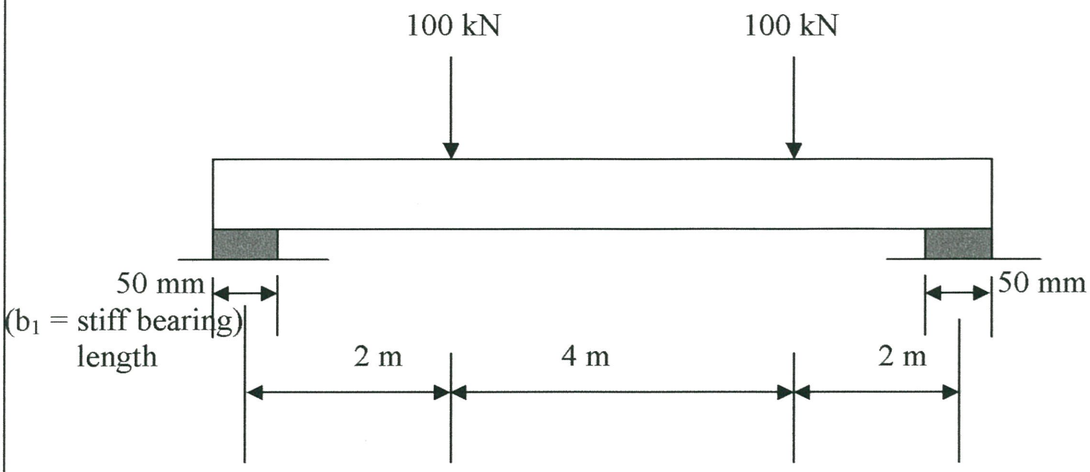
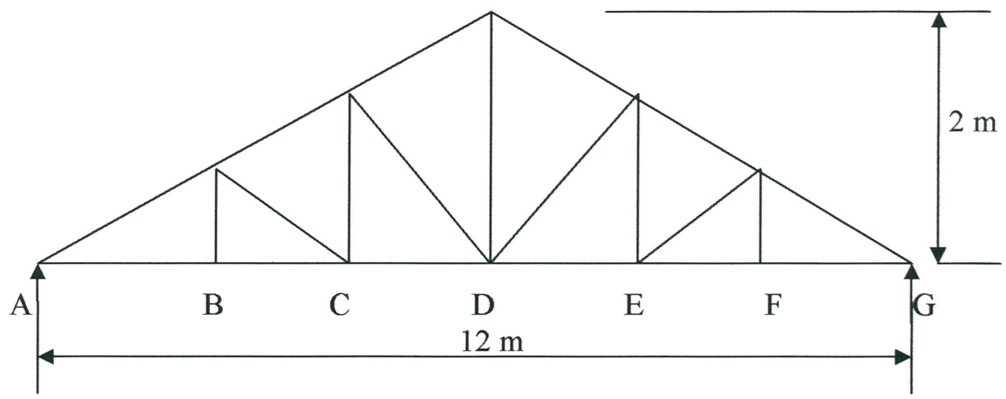


Figure Q4



AB = BC=CD=DE=EF=FG

Figure Q5

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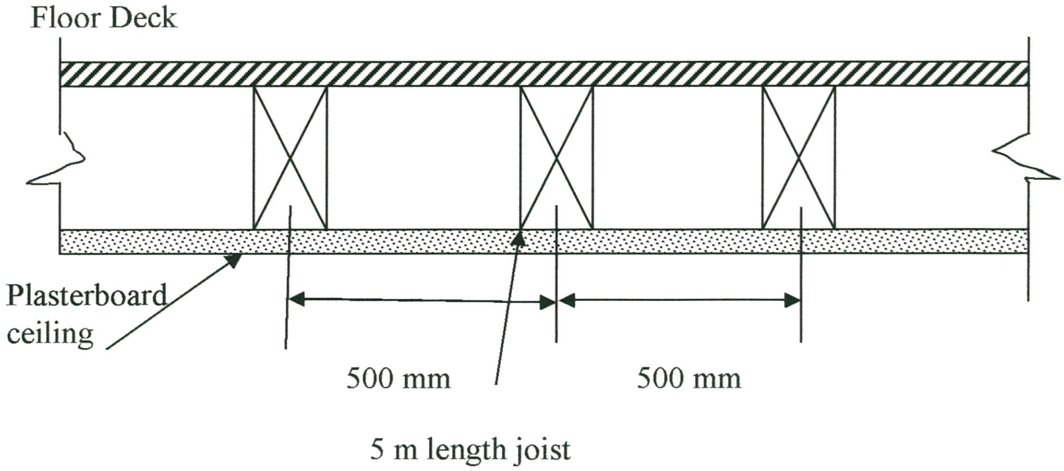

Figure Q6

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APPENDIX 1

Table 1: Cross-sectional area according to size and number (This table is used in reinforced concrete beam and column design)

Bar size	Number of bars								
	1	2	3	4	5	6	7	8	9
H6	28	57	85	113	141	170	198	226	254
H8	50	101	151	201	251	302	352	402	452
H10	79	157	236	314	393	471	550	628	707
H12	113	226	339	452	565	679	792	905	1018
H16	201	402	603	804	1005	1206	1407	1608	1810
H20	314	628	942	1257	1571	1885	2199	2513	2827
H25	491	982	1473	1963	2454	2945	3436	3927	4418
H32	804	1608	2413	3217	4021	4825	5630	6434	7238
H40	1257	2513	3770	5027	6283	7540	8796	10053	11310

Table 2: This table is used when designing reinforced concrete slab

Bar size	Spacing of bars (mm)								
	75	100	125	150	175	200	250	300	350
H6	377	283	226	188	162	141	113	94	81
H8	670	503	402	335	287	251	201	168	144
H10	1047	785	628	524	449	393	314	262	224
H12	1508	1131	905	754	646	565	452	377	323
H16	2681	2011	1608	1340	1149	1005	804	670	574
H20	4189	3142	2513	2094	1795	1571	1257	1047	898
H25	6545	4909	3927	3272	2805	2454	1963	1636	1402
H32	10723	8042	6434	5362	4596	4021	3217	2681	2298
H40	16755	12566	10053	8378	7181	6283	5027	4189	3590

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Design Formula For Beam

$K = M/bd^2f_{ck} < 0.167$

$$z = \frac{d}{2}\left[1 + \sqrt{1 - 3.53K}\right] \leq 0.95d$$

$$A_s = \frac{M_d}{0.87 f_{yk} z}$$

If $K > 0.167$

$$z = \frac{d}{2}\left[1 + \sqrt{1 - 3.53K'}\right]$$

$$A_s' = \frac{(K - K')f_{ck}bd^2}{0.87 f_{yk}(d - d')}$$

$$A_s = \frac{K' f_{ck} bd^2}{0.87 f_{yk} z_{bal}} + A_s'$$

Two ways slab

$m_{sx} = \alpha_{sx} n l_x^2$ .... equat. 1
$m_{sy} = \alpha_{sY} n l_x^2$ ....equat. 2

where α_{sx} and α_{sY} are given below.
n – total design ultimate load per unit area ($1.35G_k + 1.5\ Q_k$)
l_x- length of shorter side

Table 3: Bending coefficients for slab spanning in two directions at right angle, simply supported on all 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

$A_{sx} = M_{sx}/0.87f_{yk}z$, $A_{sy} = M_{sy}/0.87f_{yk}z$ per metre width

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APPENDIX 2

Table 4: Minimum percentage of tensile reinforcement in beams and slabs

Conc. strength f_{ck} (N/mm^2)	25	28	30	32	35	40	45	50
Minimum % of reinforcement	0.14	0.15	0.15	0.16	0.17	0.19	0.20	0.22

This table uses $0.016 f_{ck}^{2/3}$ as recommended by the *IStructE manual for the design of concrete building structures to Eurocode 2* in place of $0.0156 f_{ck}^{2/3}$ in EC2.

Table 5: Maximum bar spacing: other provisions

Location	Maximum bar spacing
Reinforcement in slabs 200 mm thick or less	Main reinforcement, **3 *d*** but **not more than 400 mm** Secondary reinforcement, 3.5 *d* but not more than 450 mm

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APPENDIX 3

Concrete Column Design

Limiting l_0/b ratio = $\mathbf{6.19\sqrt{(bhf_{ck}/N_{Ed})}}$

Table 6 : Limits on longitudinal reinforcement in columns

Area of longitudinal reinforcement.	Not less than or Not more than or	$0.12N/f_{yk}$ $0.002bh$ $0.04bh$ generally $0.08bh$ at laps
Size of longitudinal reinforcement	Not less than	12 mm
Number of longitudinal bars	Not fewer than	4 in a square column 6 in a circular column

Table 7: Limits on ties in reinforced concrete columns

Size of ties	Not less than or	main bar size/4 6 mm
Spacing of ties generally	Not less than or or	20 × main bar diameter the least column dimension 400 mm
Spacing of ties over a height equal to the larger dimension of the column above or below a beam or slab	Not more than or or	12 × main bar diameter 0.6 × the least column dimension 240 mm
Spacing of ties where longitudinal bars exceeding 12 mm in size are lapped	Not more than or or	12 × main bar diameter 0.6 × the least column dimension 240 mm

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APPENDIX 4

Check for bi-axial bending:

$$\frac{e_z}{h} / \frac{e_y}{b} \geq 0.2 \quad \text{and} \quad \frac{e_y}{b} / \frac{e_z}{h} \geq 0.2$$

(a) if $\frac{M_z}{h'} \geq \frac{M_y}{b'}$

then the increased single axis design moment is

$$M'_z = M_z + \beta \frac{h'}{b'} \times M_y$$

(b) if $\frac{M_z}{h'} < \frac{M_y}{b'}$

then the increased single axis design moment is

$$M'_y = M_y + \beta \frac{b'}{h'} \times M_z$$

$\frac{N_{ED}}{bhf_{ck}}$	0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	≥0.75
β	1.00	0.91	0.81	0.72	0.63	0.53	0.44	0.35	0.3

Table 8: Longitudinal reinforcement in braced rectangular columns with axial load

N/bhf_{ck}	up to 0.45	0.5	0.6	0.7	0.8	0.9	1
A_s/bhf_{ck}	0	0.17×10^{-3}	0.51×10^{-3}	0.85×10^{-3}	1.19×10^{-3}	1.53×10^{-3}	1.87×10^{-3}

Based on the design chart for $d_2/h = 0.25$ and a nominal eccentricity of $0.1h$ with $f_{yk} = 500$ N/mm^2.

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APPENDIX 5

CHART NO 1

Column design chart for rectangular columns $d_2/h = 0.05$

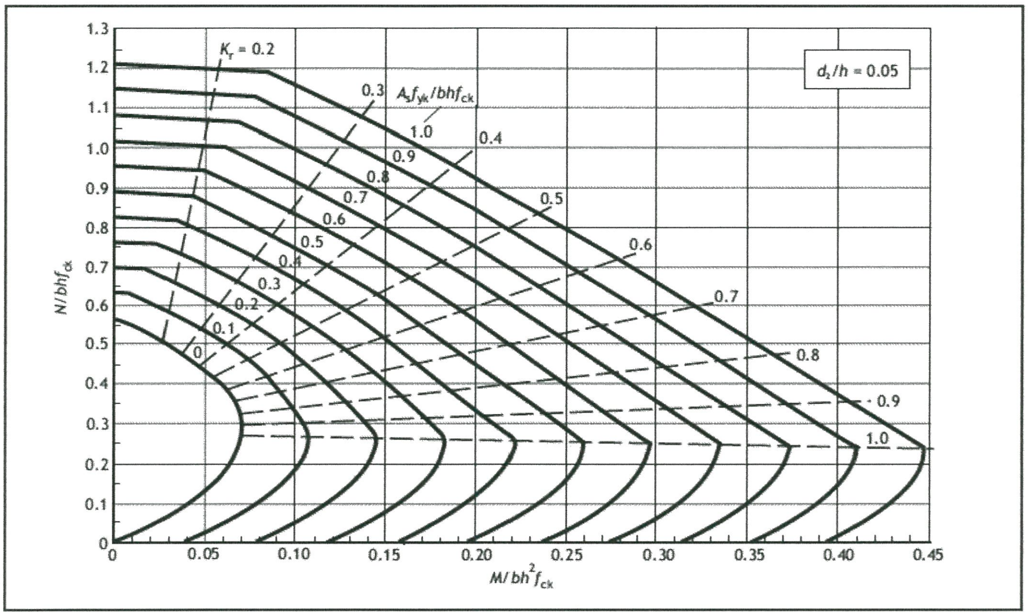
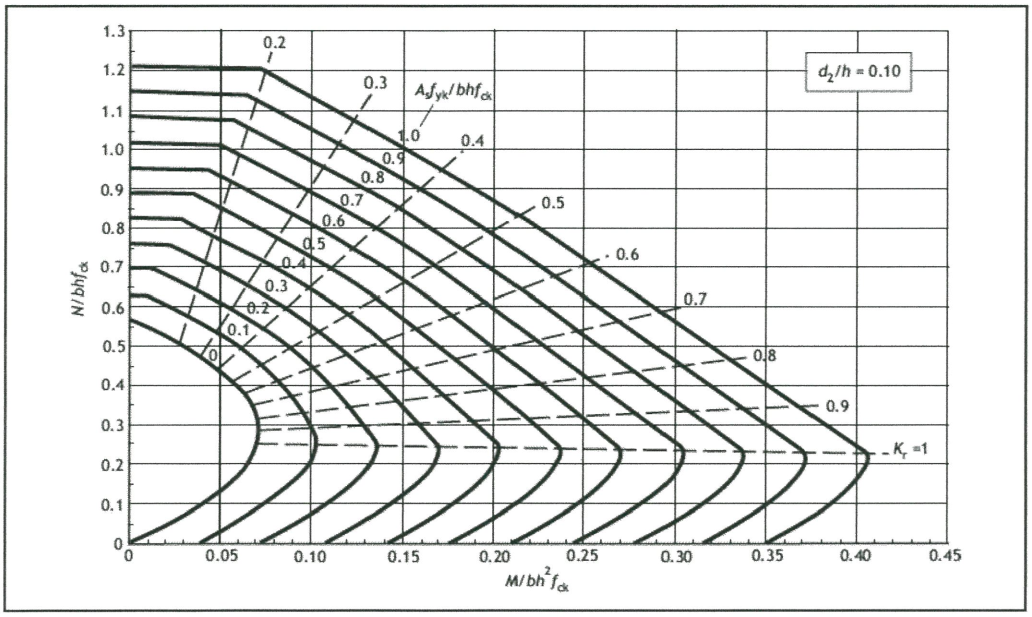


CHART NO 2

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



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APPENDIX 6

CHART NO 3

Column design chart for rectangular columns $d_2/h = 0.15$

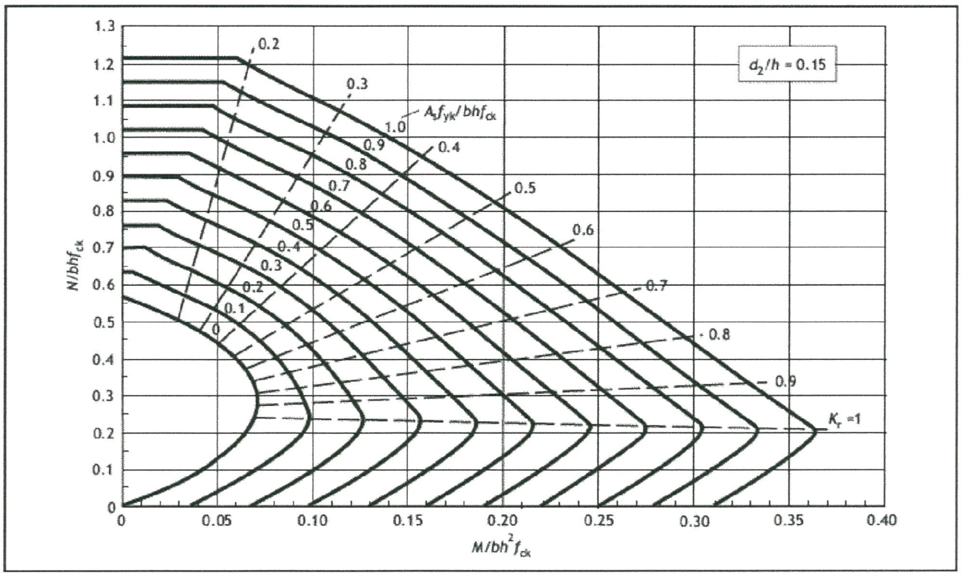
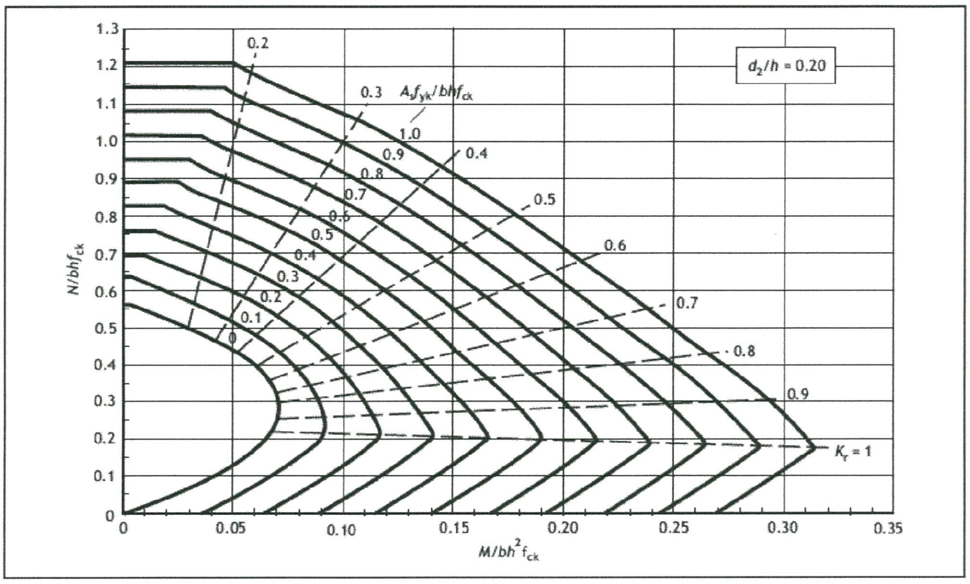


CHART NO 4

Column design chart for rectangular columns $d_2/h = 0.20$



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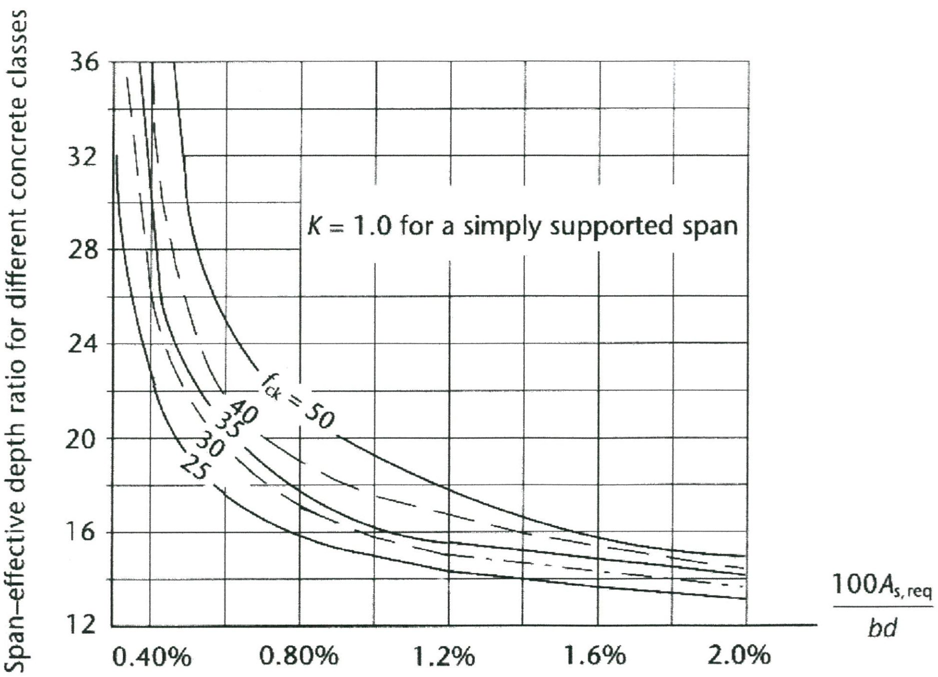
APPENDIX 7

DEFLECTION



Graph of basic span-effective depth ratios for different classes of concrete

Allowable deflection= L/d x K_1 x K_2 x K_3

$K_1 = 1$, $K_2 = 1$, $K_3 = \frac{A_{s,prov}}{A_{s,req}}$ (upper limit = 1.5)

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APPENDIX 8

Steel Beam Design

Table 8: Limiting width-to-thickness ratios for sections other than CHS and RHS

Types of element (all rolled sections)	Class of Section		
	Plastic	Compact	Semi-compact
Outstand element of compression flange	$b/T \leq 9\varepsilon$	$b/T \leq 10\varepsilon$	$b/T \leq 15\varepsilon$
Web with neutral axis at mid-depth	$d/t \leq 80\varepsilon$	$d/t \leq 100\varepsilon$	$d/t \leq 120\varepsilon$

Where $\varepsilon = (275/p_y)^{1/2}$

Clause 4.2.3 : The shear force, F_v, should not exceed the shear capacity, P_v

$$F_v \leq P_v \quad \text{................(1)}$$

Where $P_v = 0.6\, p_y A_v$ ....(2) $A_v = tD$

Clause 4.2.5.2: Low shear and Moment Capacity

If shear force $F_v < 0.6P_v$,

For Class 1 Plastic or Class 2 compact, the moment capacity is:

$$M_c = p_y S \leq 1.2\, p_y Z_x \text{ .... (3)}$$

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APPENDIX 9

Clause 4.5.2.1: The bearing resistance P_{bw} calculated at the flange/web connection for unstiffened web as:

$$P_{bw} = (b_1 + nk)\, tp_{yw} \quad \ldots\ldots\ldots\ldots\ldots (4)$$

Where:

b_1 _ stiff bearing length
n n =5 except at the end of a member and $n = 2 + 0.6b_e/k \leq 5$ at the end of the member.
b_e Distance to the end of the member from the end of the stiff bearing
k = (T + r) from rolled I or H sections
T thickness of the flange
t web thickness
p_{yw} design strength of web

Clause 4.5.3.1: Buckling resistance of Unstiffened web

If the distance a_e from the load or reaction to the nearer end of the member is less than $0.7d$, the buckling resistance P_x of the web should be taken as:

$$P_x = \frac{a_e + 0.7d}{1.4d} \frac{25\varepsilon t}{\sqrt{(b_1 + nk)d}} P_{bw}$$

Truss Design

Table 27 — Empirical values for purlins

Purlin section	Z_p (cm³)	Z_q (cm³)		D (mm)	B (mm)
		Wind load from BS 6399-2	Wind load from CP3:Ch V:Part 2		
Angle	$W_pL/1\,800$	$W_qL/2\,250$	$W_qL/1\,800$	$L/45$	$L/60$
CHS	$W_pL/2\,000$	$W_qL/2\,500$	$W_qL/2\,000$	$L/65$	—
RHS	$W_pL/1\,800$	$W_qL/2\,250$	$W_qL/1\,800$	$L/70$	$L/150$

NOTE 1 W_p and W_q are the total unfactored loads (in kN) on one span of the purlin, acting perpendicular to the plane of the cladding, due to (dead plus imposed) and (wind minus dead) loading respectively.

NOTE 2 L is the span of the purlin (in mm) centre-to-centre of main vertical supports. However, if properly supported sag rods are used, L may be taken as the sag rod spacing in determining B only.

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APPENDIX 10

Clause 4.6.3 Tension capacity for simple tensión members:

— for bolted connections: $P_t = p_y(A_e - 0.5a_2)$

— for welded connections: $P_t = p_y(A_g - 0.3a_2)$

$$a_2 = A_g - a_1$$

A_g is the gross cross-sectional area, see **3.4.1**;

a_1 is the gross area of the connected element, taken as the product of its thickness and the overall leg width for an angle, the overall depth for a channel or the flange width for a T-section.