

Engineering Services (Main)
Examination - 2025

NESP-O-CVLE

CIVIL ENGINEERING

PAPER—I

Time Allowed : Three Hours

Maximum Marks : 300

QUESTION PAPER SPECIFIC INSTRUCTIONS

Please read each of the following instructions carefully before attempting questions

There are **EIGHT** questions divided in **TWO** Sections.

Candidate has to attempt **FIVE** questions in all.

Question Nos. **1** and **5** are **compulsory** and out of the remaining, any **THREE** are to be attempted choosing at least **ONE** question from each Section.

The number of marks carried by a question/part is indicated against it.

Wherever any assumptions are made for answering a question, they must be clearly indicated.

Diagrams/Figures, wherever required, shall be drawn in the space provided for answering the question itself.

Unless otherwise mentioned, symbols and notations have their usual standard meanings.

Attempts of questions shall be counted in sequential order. Unless struck off, attempt of a question shall be counted even if attempted partly.

Any page or portion of the page left blank in the Question-cum-Answer Booklet must be clearly struck off.

Answers must be written in **ENGLISH** only.

SECTION—A

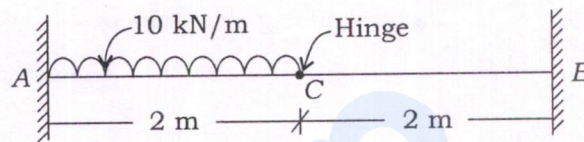
1. (a) (i) Write the qualities of good timber and the factors affecting the strength of timber. 6

(ii) Write short notes on any *three* of the following :

- (1) Brick buttresses
- (2) Brick corbel
- (3) Brick coping
- (4) Thresholds
- (5) Brick jambs
- (6) Racking back

2×3=6

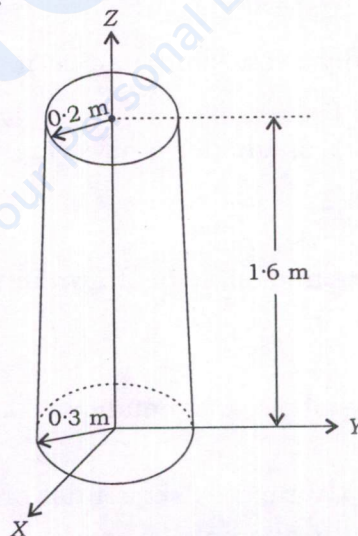
- (b) Draw the bending moment and shear force diagram for the beam shown below :



12

- (c) With the help of a sketch, briefly explain the major shear transfer mechanisms in a reinforced concrete beam, having shear reinforcement. Indicate the internal forces acting at a flexural-shear crack. 12

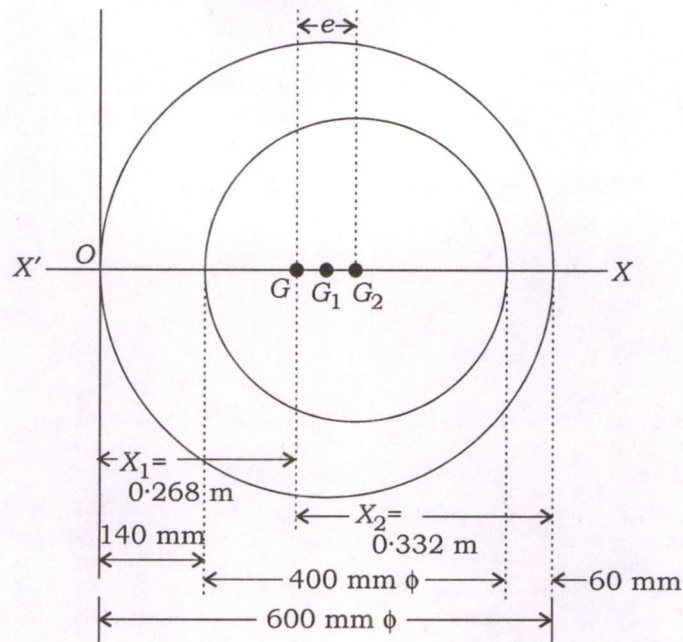
- (d) A pedestal in the shape of a frustum of a cone is made of concrete having a specific weight of 24 kN/m^3 . Determine the average normal stress acting in the pedestal at its base :



12

- (e) A four-wheel tractor weighing 18000 kg has weight distribution between the front and the rear wheels of 40 percent and 60 percent respectively. It is operating on a level haul road whose rolling resistance is 45 kg/ton. What is the maximum rimpull of the tractor if the coefficient of traction between the road surface and the tyre is 0.65? 12

2. (a)



A short hollow cast iron column having an external diameter of 600 mm and inside diameter 400 mm was cast in a factory. On inspection, it was found that the bore is eccentric as shown in the figure above. If the column carries a load of 2000 kN along the axis of the bore, calculate the extreme intensities of stresses induced in the section.

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(b) Explain the following terms :

- (i) Autogenous shrinkage
- (ii) Bogue compounds
- (iii) Case-hardening
- (iv) Elastomers
- (v) Guniting
- (vi) Scoriaceous aggregate
- (vii) Self-desiccation
- (viii) Shingling
- (ix) Puddling
- (x) Wet rot

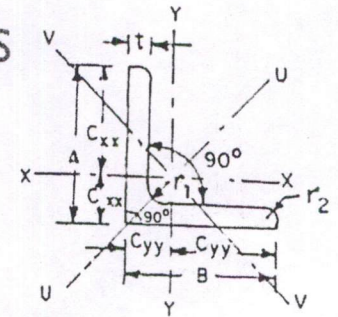
2×10=20

(c) Design a suitable double-angle discontinuous strut in a steel truss to carry a working axial compressive load of 200 kN. The effective length of the strut is 2.12 m. Use a gusset plate of 20 mm thick. Assume column buckling class c. $f_y = 250$ MPa, $f_u = 400$ MPa, $\gamma_{mb} = 1.25$.

Relevant portion of the code books is enclosed.

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TABLE III ROLLED STEEL EQUAL ANGLES
DIMENSIONS AND PROPERTIES
 (Continued)



Designation	Size		Thickness <i>t</i>	Sectional Area <i>a</i>	Weight per Metre <i>w</i>	Centre of Gravity $C_{xx} = C_{yy}$	Distance of Extreme Fibre $e_{xx} = e_{yy}$
	<i>A</i> × <i>B</i>						
	mm	mm	mm	cm ²	kg	cm	cm
ISA 7070	70 × 70		5.0	6.77	5.3	1.89	5.11
			6.0	8.06	6.3	1.94	5.06
			8.0	10.58	8.3	2.02	4.98
			10.0	13.02	10.2	2.10	4.90
ISA 7575	75 × 75		5.0	7.27	5.7	2.02	5.48
			6.0	8.66	6.8	2.06	5.44
			8.0	11.38	8.9	2.14	5.36
			10.0	14.02	11.0	2.22	5.28
ISA 8080	80 × 80		6.0	9.29	7.3	2.18	5.82
			8.0	12.21	9.6	2.27	5.73
			10.0	15.05	11.8	2.34	5.66
			12.0	17.81	14.0	2.42	5.58
ISA 9090	90 × 90		6.0	10.47	8.2	2.42	6.58
			8.0	13.79	10.8	2.51	6.49
			10.0	17.03	13.4	2.59	6.41
			12.0	20.19	15.8	2.66	6.34
ISA 100100	100 × 100		6.0	11.67	9.2	2.67	7.33
			8.0	15.39	12.1	2.76	7.24
			10.0	19.03	14.9	2.84	7.16
			12.0	22.59	17.7	2.92	7.08
ISA 110110	110 × 110		8.0	17.02	13.4	3.00	8.00
			10.0	21.06	16.5	3.08	7.92
			12.0	25.02	19.6	3.16	7.84
			15.0	30.81	24.2	3.27	7.73
ISA 130130	130 × 130		8.0	20.22	15.9	3.50	9.50
			10.0	25.06	19.7	3.58	9.42
			12.0	29.82	23.4	3.66	9.34
			15.0	36.81	28.9	3.78	9.22
ISA 150150	150 × 150		10.0	29.03	22.8	4.06	10.94
			12.0	34.59	27.2	4.14	10.86
			15.0	42.78	33.6	4.26	10.74
			18.0	50.79	39.9	4.38	10.62
ISA 200200	200 × 200		12.0	46.61	36.6	5.36	14.64
			15.0	57.80	45.4	5.49	14.51
			18.0	68.81	54.0	5.61	14.39
			25.0	93.80	73.6	5.88	14.12

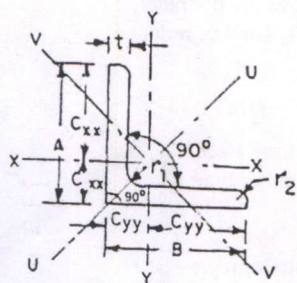


TABLE III ROLLED STEEL EQUAL ANGLES

DIMENSIONS AND PROPERTIES

(Continued)

Moments of Inertia			Radii of Gyration			Modulus of Section	Radius at Root	Radius at Toe	Product of Inertia	Designation
$I_{xx} = I_{yy}$	I_{uu}	I_{vv}	$r_{xx} = r_{yy}$	r_{uu}	r_{vv}	$Z_{xx} = Z_{yy}$	r_1	r_2	I_{xy}	
cm ⁴	cm ⁴	cm ⁴	cm	cm	cm	cm ³	mm	mm	cm ⁴	
31.1	49.8	12.5	2.15	2.71	1.36	6.1	7.0	4.5	18.4	ISA 7070
36.8	58.8	14.8	2.14	2.70	1.36	7.3			21.7	
47.4	75.5	19.3	2.12	2.67	1.35	9.5			27.9	
57.2	90.7	23.7	2.10	2.64	1.35	11.7			33.3	
38.7	61.9	15.5	2.31	2.92	1.46	7.1	7.0	4.5	22.8	ISA 7575
45.7	73.1	18.4	2.30	2.91	1.46	8.4			27.0	
59.0	94.1	24.0	2.28	2.88	1.45	11.0			34.8	
71.4	113.3	29.4	2.26	2.84	1.45	13.5			41.7	
56.0	89.6	22.5	2.46	3.11	1.56	9.6	8.0	4.5	33.0	ISA 8080
72.5	115.6	29.4	2.44	3.08	1.55	12.6			42.7	
87.7	139.5	36.0	2.41	3.04	1.55	15.5			51.4	
101.9	161.4	42.4	2.39	3.01	1.54	18.3			59.2	
80.1	128.1	32.0	2.77	3.50	1.75	12.2	8.5	5.5	47.2	ISA 9090
104.2	166.4	42.0	2.75	3.47	1.75	16.0			61.5	
126.7	201.9	51.6	2.73	3.44	1.74	19.8			74.5	
147.9	234.9	60.9	2.71	3.41	1.74	23.3			86.5	
111.3	178.1	44.5	3.09	3.91	1.95	15.2	8.5	5.5	65.7	ISA 100100
145.1	231.8	58.4	3.07	3.88	1.95	20.0			85.8	
177.0	282.2	71.8	3.05	3.85	1.94	24.7			104.4	
207.0	329.3	84.7	3.03	3.82	1.94	29.2			121.6	
195.0	311.7	78.2	3.38	4.28	2.14	24.4	10.0	6.0	115.1	ISA 110110
238.4	380.5	96.3	3.36	4.25	2.14	30.1			140.6	
279.6	445.3	113.8	3.34	4.22	2.13	35.7			164.5	
337.4	535.4	139.3	3.31	4.17	2.13	43.7			197.0	
328.3	525.1	131.4	4.03	5.10	2.55	34.5	10.0	6.0	194.2	ISA 130130
402.7	643.4	162.1	4.01	5.07	2.54	42.7			238.3	
473.8	755.9	191.8	3.99	5.03	2.54	50.7			279.9	
574.6	914.2	235.0	3.95	4.98	2.53	62.3			337.8	
622.4	995.4	249.4	4.63	5.86	2.93	56.9	12.0	8.0	368.2	ISA 150150
735.4	1174.8	296.0	4.61	5.83	2.93	67.7			435.0	
896.8	1429.7	363.8	4.58	5.78	2.92	83.5			529.1	
1048.9	1668.2	429.5	4.54	5.73	2.91	98.7			616.0	
1788.9	2862.0	715.9	6.20	7.84	3.92	122.2	15.0	10.0	1058.9	ISA 200200
2197.7	3511.8	883.7	6.17	7.79	3.91	151.4			1301.2	
2588.7	4130.8	1046.5	6.13	7.75	3.90	179.9			1530.5	
3436.3	5460.9	1411.6	6.05	7.63	3.88	243.3			2015.7	

IS 800 : 2007

10.3.1.3 In the calculation of thread length, allowance should be made for tolerance and thread run off.

10.3.2 A bolt subjected to a factored shear force (V_{sb}) shall satisfy the condition

$$V_{sb} = V_{db}$$

where V_{db} is the design strength of the bolt taken as the smaller of the value as governed by shear, V_{dsb} (see 10.3.3) and bearing, V_{dpb} (see 10.3.4).

10.3.3 Shear Capacity of Bolt

The design strength of the bolt, V_{dsb} as governed shear strength is given by:

$$V_{dsb} = V_{nsb} / \gamma_{mb}$$

where

V_{nsb} = nominal shear capacity of a bolt, calculated as follows:

$$V_{nsb} = \frac{f_u}{\sqrt{3}} (n_n A_{nb} + n_s A_{sb})$$

where

f_u = ultimate tensile strength of a bolt;

n_n = number of shear planes with threads intercepting the shear plane;

n_s = number of shear planes without threads intercepting the shear plane;

A_{sb} = nominal plain shank area of the bolt; and

A_{nb} = net shear area of the bolt at threads, may be taken as the area corresponding to root diameter at the thread.

10.3.3.1 Long joints

When the length of the joint, l_j of a splice or end connection in a compression or tension element containing more than two bolts (that is the distance between the first and last rows of bolts in the joint, measured in the direction of the load transfer) exceeds $15d$ in the direction of load, the nominal shear capacity (see 10.3.2), V_{db} shall be reduced by the factor β_{lj} , given by:

$$\beta_{lj} = 1.075 - l_j / (200 d) \text{ but } 0.75 \leq \beta_{lj} \leq 1.0$$

$$= 1.075 - 0.005(l_j/d)$$

where

d = Nominal diameter of the fastener.

NOTE — This provision does not apply when the distribution of shear over the length of joint is uniform, as in the connection of web of a section to the flanges.

10.3.3.2 Large grip lengths

When the grip length, l_g (equal to the total thickness of

the connected plates) exceeds 5 times the diameter, d of the bolts, the design shear capacity shall be reduced by a factor β_{lg} , given by:

$$\beta_{lg} = 8 d / (3 d + l_g) = 8 / (3 + l_g/d)$$

β_{lg} shall not be more than β_{lj} given in 10.3.3.1. The grip length, l_g shall in no case be greater than $8d$.

10.3.3.3 Packing plates

The design shear capacity of bolts carrying shear through a packing plate in excess of 6 mm shall be decreased by a factor, β_{pk} given by:

$$\beta_{pk} = (1 - 0.0125 t_{pk})$$

where

t_{pk} = thickness of the thicker packing, in mm.

10.3.4 Bearing Capacity of the Bolt

The design bearing strength of a bolt on any plate, V_{dpb} as governed by bearing is given by:

$$V_{dpb} = V_{npb} / \gamma_{mb}$$

where

$$V_{npb} = \text{nominal bearing strength of a bolt}$$

$$= 2.5 k_b d t f_u$$

where

$$k_b \text{ is smaller of } \frac{e}{3d_0}, \frac{p}{3d_0} - 0.25, \frac{f_{ub}}{f_u}, 1.0;$$

e, p = end and pitch distances of the fastener along bearing direction;

d_0 = diameter of the hole;

f_{ub}, f_u = ultimate tensile stress of the bolt and the ultimate tensile stress of the plate, respectively;

d = nominal diameter of the bolt; and

t = summation of the thicknesses of the connected plates experiencing bearing stress in the same direction, or if the bolts are countersunk, the thickness of the plate minus one half of the depth of countersinking.

The bearing resistance (in the direction normal to the slots in slotted holes) of bolts in holes other than standard clearance holes may be reduced by multiplying the bearing resistance obtained as above, V_{npb} , by the factors given below:

- Over size and short slotted holes — 0.7, and
- Long slotted holes — 0.5.

NOTE — The block shear of the edge distance due to bearing force may be checked as given in 6.4.

Table 9(c) Design Compressive Stress, f_{cd} (MPa) for Column Buckling Class c
(Clause 7.1.2.1)

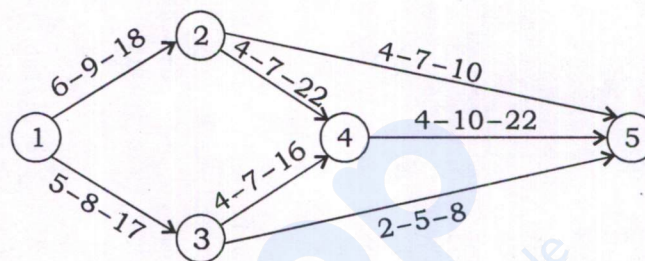
KL/r ↓	Yield Stress, f_y (MPa)																			
	200	210	220	230	240	250	260	280	300	320	340	360	380	400	420	450	480	510	540	
10	182	191	200	209	218	227	236	255	273	291	309	327	345	364	382	409	436	464	491	
20	182	190	199	207	216	224	233	250	266	283	299	316	332	348	364	388	412	435	458	
30	172	180	188	196	204	211	219	234	249	264	278	293	307	321	335	355	376	395	415	
40	163	170	177	184	191	198	205	218	231	244	256	268	280	292	304	320	337	352	367	
50	153	159	165	172	178	183	189	201	212	222	232	242	252	261	270	282	295	306	317	
60	142	148	153	158	163	168	173	182	191	199	207	215	222	228	235	244	252	260	267	
70	131	136	140	144	148	152	156	163	170	176	182	187	192	197	202	208	213	218	223	
80	120	123	127	130	133	136	139	145	149	154	158	162	165	169	172	176	180	183	186	
90	108	111	114	116	119	121	123	127	131	134	137	140	142	144	146	149	152	154	156	
100	97.5	100	102	104	105	107	109	112	114	116	119	120	122	124	125	127	129	131	132	
110	87.3	89.0	90.5	92.0	93.3	94.6	95.7	97.9	100	102	103	104	106	107	108	110	111	112	113	
120	78.2	79.4	80.6	81.7	82.7	83.7	84.6	86.2	87.6	88.9	90.1	91.1	92.1	93.0	93.8	94.9	95.9	96.8	97.6	
130	70.0	71.0	71.9	72.8	73.5	74.3	75.0	76.2	77.3	78.3	79.2	80.0	80.7	81.4	82.0	82.9	83.6	84.3	84.9	
140	62.9	63.6	64.4	65.0	65.6	66.2	66.7	67.7	68.6	69.3	70.0	70.7	71.2	71.8	72.3	72.9	73.5	74.1	74.6	
150	56.6	57.2	57.8	58.3	58.8	59.2	59.7	60.4	61.1	61.7	62.3	62.8	63.3	63.7	64.1	64.6	65.1	65.5	65.9	
160	51.1	51.6	52.1	52.5	52.9	53.3	53.6	54.2	54.8	55.3	55.7	56.1	56.5	56.9	57.2	57.6	58.0	58.4	58.7	
170	46.4	46.8	47.1	47.5	47.8	48.1	48.4	48.9	49.3	49.8	50.1	50.5	50.8	51.1	51.3	51.7	52.0	52.3	52.6	
180	42.2	42.5	42.8	43.1	43.4	43.6	43.9	44.3	44.7	45.0	45.3	45.6	45.8	46.1	46.3	46.6	46.9	47.1	47.3	
190	38.5	38.8	39.0	39.3	39.5	39.7	39.9	40.3	40.6	40.9	41.1	41.4	41.6	41.8	42.0	42.2	42.5	42.7	42.9	
200	35.3	35.5	35.7	35.9	36.1	36.3	36.5	36.8	37.0	37.3	37.5	37.7	37.9	38.1	38.2	38.4	38.6	38.8	39.0	
210	32.4	32.6	32.8	33.0	33.1	33.3	33.4	33.7	33.9	34.1	34.3	34.5	34.7	34.8	34.9	35.1	35.3	35.4	35.6	
220	29.9	30.1	30.2	30.4	30.5	30.6	30.8	31.0	31.2	31.4	31.5	31.7	31.8	31.9	32.1	32.2	32.4	32.5	32.6	
230	27.6	27.8	27.9	28.0	28.2	28.3	28.4	28.6	28.8	28.9	29.1	29.2	29.3	29.4	29.5	29.7	29.8	29.9	30.0	
240	25.6	25.7	25.9	26.0	26.1	26.2	26.3	26.4	26.6	26.7	26.9	27.0	27.1	27.2	27.3	27.4	27.5	27.6	27.7	
250	23.8	23.9	24.0	24.1	24.2	24.3	24.4	24.5	24.7	24.8	24.9	25.0	25.1	25.2	25.3	25.4	25.5	25.6	25.7	

3. (a) For the network shown below, the time estimates (in days) for each activity are mentioned. Determine the probability of completing the project in 35 days.

Given :

Standard normal distribution function

Z	P% (% probability)
0.8	78.51
0.9	81.59
1.0	84.13
1.1	86.43
1.2	88.49



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- (b) Design a short reinforced concrete column subjected to a working axial load of 1400 kN and service moments of 60 kN-m and 40 kN-m about its major and minor axes respectively. The least cross-sectional dimension of the column shall be 300 mm. Adopt limit state design. Use M 30 concrete and Fe 500 grade steel. The effective concrete cover to longitudinal reinforcement is 60 mm. Sketch the reinforcement details.

Relevant portions of IS 456 and SP 16 are enclosed.

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IS 456 : 2000

39.3 Short Axially Loaded Members in Compression

The member shall be designed by considering the assumptions given in 39.1 and the minimum eccentricity. When the minimum eccentricity as per 25.4 does not exceed 0.05 times the lateral dimension, the members may be designed by the following equation:

$$P_u = 0.4 f_{ck} A_c + 0.67 f_y A_{sc}$$

where

- P_u = axial load on the member,
 f_{ck} = characteristic compressive strength of the concrete,
 A_c = Area of concrete,
 f_y = characteristic strength of the compression reinforcement, and
 A_{sc} = area of longitudinal reinforcement for columns.

39.4 Compression Members with Helical Reinforcement

The strength of compression members with helical reinforcement satisfying the requirement of 39.4.1 shall be taken as 1.05 times the strength of similar member with lateral ties.

39.4.1 The ratio of the volume of helical reinforcement to the volume of the core shall not be less than $0.36 (A_g/A_c - 1) f_{ck}/f_y$,

where

- A_g = gross area of the section,
 A_c = area of the core of the helically reinforced column measured to the outside diameter of the helix,
 f_{ck} = characteristic compressive strength of the concrete, and
 f_y = characteristic strength of the helical reinforcement but not exceeding 415 N/mm².

39.5 Members Subjected to Combined Axial Load and Uniaxial Bending

A member subjected to axial force and uniaxial bending shall be designed on the basis of 39.1 and 39.2.

NOTE — The design of member subject to combined axial load and uniaxial bending will involve lengthy calculation by trial and error. In order to overcome these difficulties interaction diagrams may be used. These have been prepared and published by BIS in 'SP : 16 Design aids for reinforced concrete to IS 456'.

39.6 Members Subjected to Combined Axial Load and Biaxial Bending

The resistance of a member subjected to axial force and biaxial bending shall be obtained on the basis of assumptions given in 39.1 and 39.2 with neutral axis so chosen as to satisfy the equilibrium of load and moments about two axes. Alternatively such members may be designed by the following equation:

$$\left[\frac{M_{ux}}{M_{ux1}} \right]^{\alpha_x} + \left[\frac{M_{uy}}{M_{uy1}} \right]^{\alpha_x} \leq 1.0$$

where

- M_{ux}, M_{uy} = moments about x and y axes due to design loads,
 M_{ux1}, M_{uy1} = maximum uniaxial moment capacity for an axial load of P_u , bending about x and y axes respectively, and

α_x is related to P_u/P_{ux}

where $P_{ux} = 0.45 f_{ck} A_c + 0.75 f_y A_{sc}$

For values of $P_u/P_{ux} = 0.2$ to 0.8, the values of α_x vary linearly from 1.0 to 2.0. For values less than 0.2, α_x is 1.0; for values greater than 0.8, α_x is 2.0.

39.7 Slender Compression Members

The design of slender compression members (see 25.1.1) shall be based on the forces and the moments determined from an analysis of the structure, including the effect of deflections on moments and forces. When the effect of deflections are not taken into account in the analysis, additional moment given in 39.7.1 shall be taken into account in the appropriate direction.

39.7.1 The additional moments M_{ux} and M_{uy} shall be calculated by the following formulae:

$$M_{ux} = \frac{P_u D}{2000} \left\{ \frac{l_{ux}}{D} \right\}^2$$

$$M_{uy} = \frac{P_u b}{2000} \left\{ \frac{l_{uy}}{b} \right\}^2$$

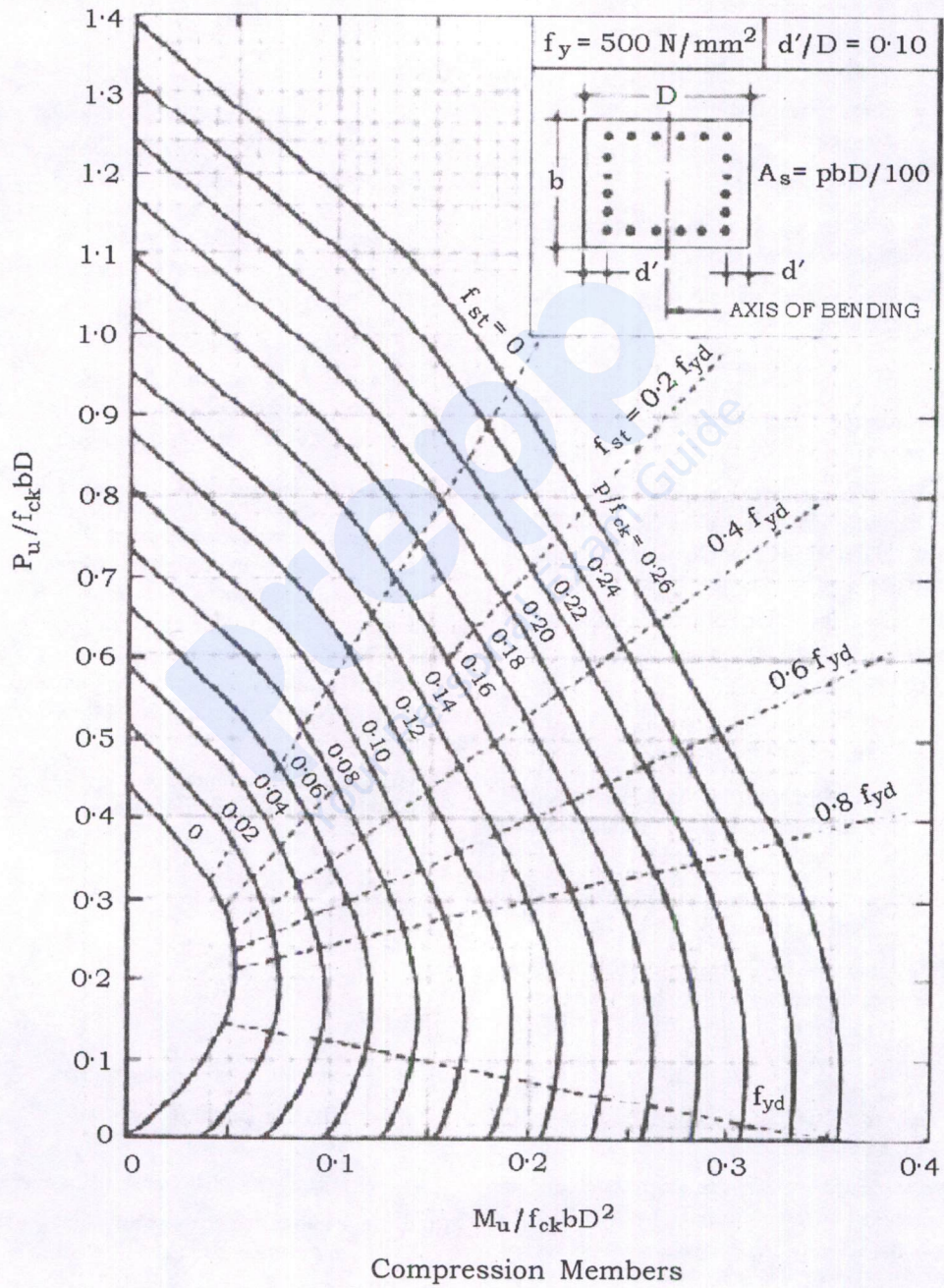
where

- P_u = axial load on the member,
 l_{ux} = effective length in respect of the major axis,
 l_{uy} = effective length in respect of the minor axis,
 D = depth of the cross-section at right angles to the major axis, and
 b = width of the member.

For design of section, 39.5 or 39.6 as appropriate shall apply.

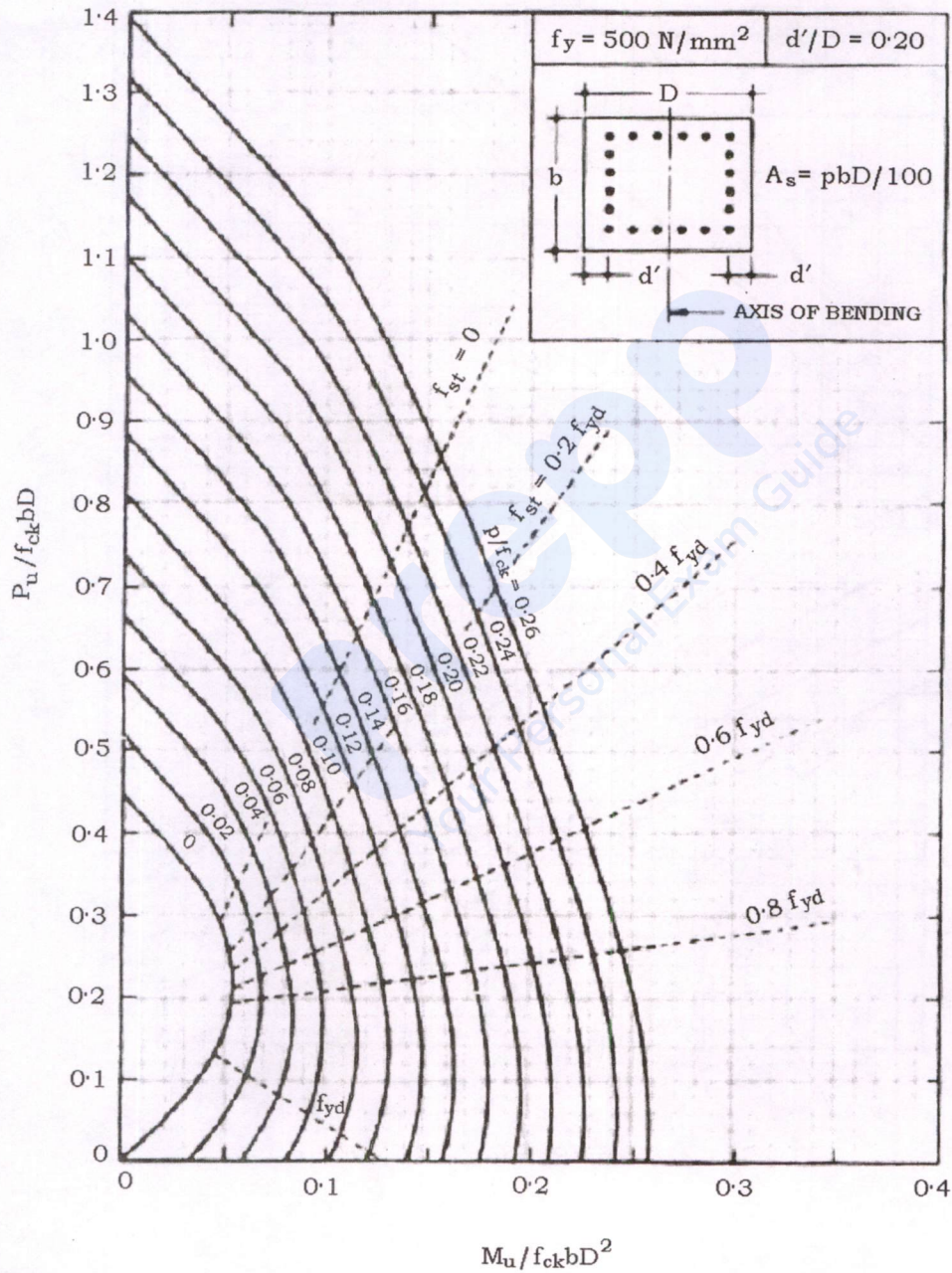
f_y
500

Chart 48 COMPRESSION WITH BENDING—Rectangular Section—Reinforcement Distributed Equally on Four Sides



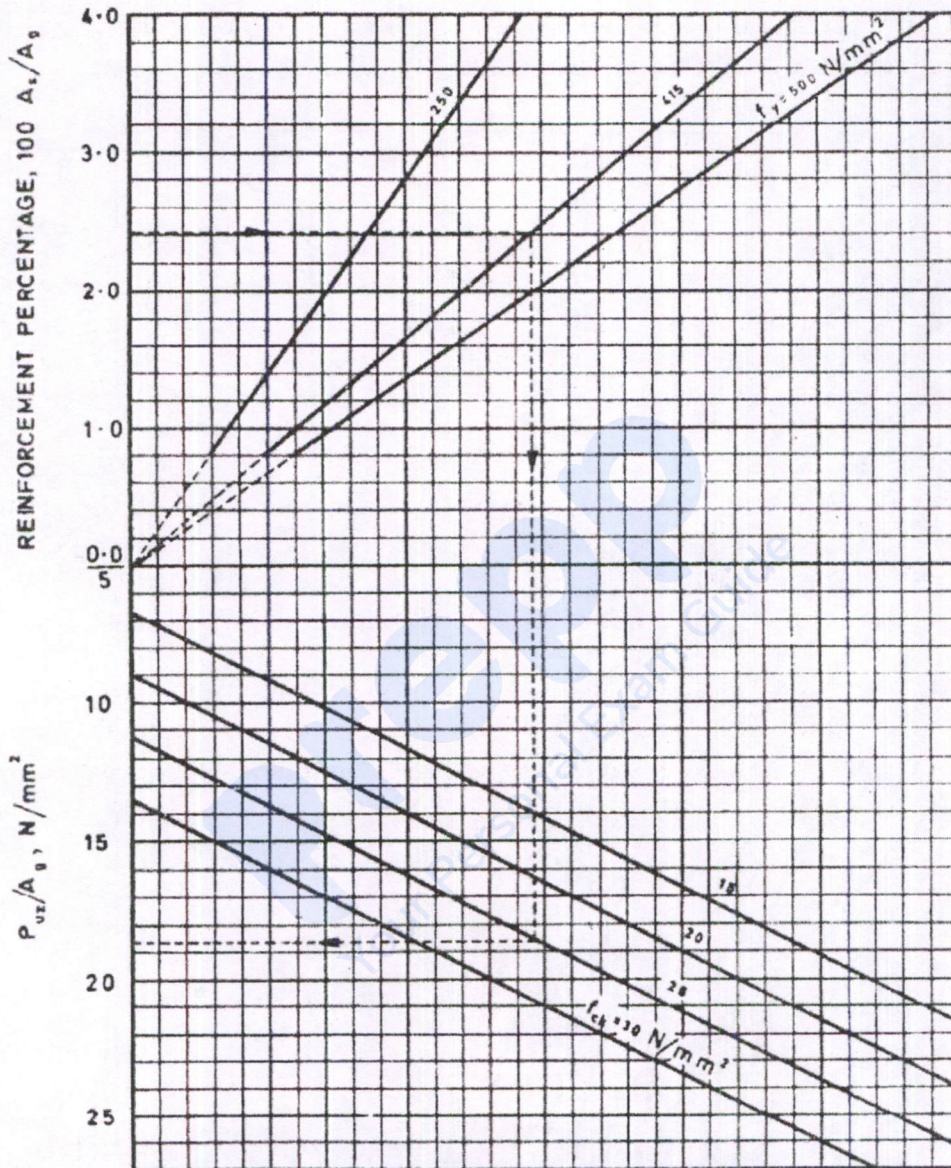
f_y
500

Chart 50 COMPRESSION WITH BENDING—Rectangular Section—Reinforcement Distributed Equally on Four Sides



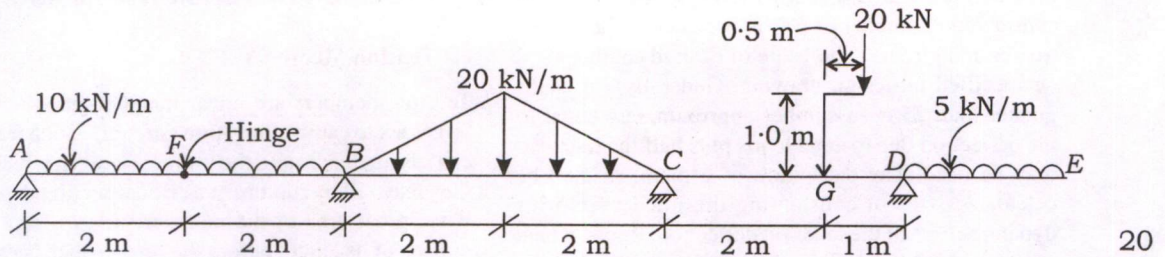
Compression Members

Chart 63 VALUES OF P_{uz} for COMPRESSION MEMBERS

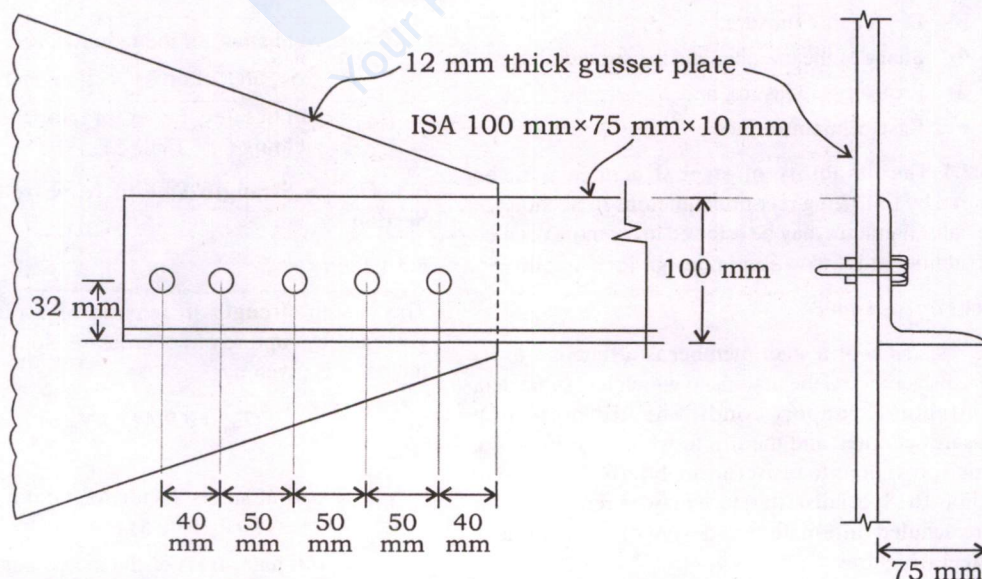


Design Aids for Reinforced Concrete

- (c) For the continuous beam shown below, draw the bending moment diagram using Clapeyron's theorem (three-moment equation). The support C sinks by 1 cm. Take $E = 200 \text{ GPa}$, $I = 10000 \text{ cm}^4$:



4. (a) Five wheel loads of 90 kN, 90 kN, 90 kN, 50 kN and 50 kN magnitudes spaced 3 m, 2 m, 1.5 m and 2 m apart respectively cross a simply supported girder of 20 m span from right to left with 90 kN load leading. Calculate the maximum positive and negative shear force at the centre of the span and absolute maximum value of bending moment that occurs anywhere in the girder. 20
- (b) The initial cost of an equipment is ₹ 1,100, salvage value is ₹ 100, life of the equipment is 5 years. The rate of interest for sinking fund is 8% . Calculate the yearly depreciation and book value at the end of each year by straight line method, declining balance method, sum of years digital method and sinking fund method. Present the value in tabular form. 20
- (c) A single angle ISA 100 mm × 75 mm × 10 mm is connected to a gusset plate of 12 mm thick with five numbers of 16 mm diameter bolts. Determine its tensile capacity if the gusset plate is connected to the 100 mm leg. The cross-sectional area of the angle is 1650 mm^2 , $f_u = 410 \text{ MPa}$, $f_y = 250 \text{ MPa}$, $\gamma_{m0} = 1.1$, $\gamma_{m1} = 1.25$.



Relevant portion of the IS 800 : 2007 is enclosed.

20

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5.6.1.1 Where the deflection due to the combination of dead load and live load is likely to be excessive, consideration should be given to pre-camber the beams, trusses and girders. The value of desired camber shall be specified in design drawing. Generally, for spans greater than 25 m, a camber approximately equal to the deflection due to dead loads plus half the live load may be used. The deflection of a member shall be calculated without considering the impact factor or dynamic effect of the loads on deflection. Roofs, which are very flexible, shall be designed to withstand any additional load that is likely to occur as a result of ponding of water or accumulation of snow or ice.

5.6.2 *Vibration*

Suitable provisions in the design shall be made for the dynamic effects of live loads, impact loads and vibration due to machinery operating loads. In severe cases possibility of resonance, fatigue or unacceptable vibrations shall be investigated. Unusually flexible structures (generally the height to effective width of lateral load resistance system exceeding 5:1) shall be investigated for lateral vibration under dynamic wind loads. Structures subjected to large number of cycles of loading shall be designed against fatigue failure, as specified in Section 13. Floor vibration effect shall be considered using specialist literature (*see Annex C*).

5.6.3 *Durability*

Factors that affect the durability of the buildings, under conditions relevant to their intended life, are listed below:

- Environment,
- Degree of exposure,
- Shape of the member and the structural detail,
- Protective measure, and
- Ease of maintenance.

5.6.3.1 The durability of steel structures shall be ensured by following recommendations in Section 15. Specialist literature may be referred to for more detailed and additional information in design for durability.

5.6.4 *Fire Resistance*

Fire resistance of a steel member is a function of its mass, its geometry, the actions to which it is subjected, its structural support condition, fire protection measures adopted and the fire to which it is exposed. Design provisions to resist fire are briefly discussed in Section 16. Specialist literature may be referred to for more detailed information in design of fire resistance of steel structures.

SECTION 6 DESIGN OF TENSION MEMBERS

6.1 *Tension Members*

Tension members are linear members in which axial forces act to cause elongation (stretch). Such members can sustain loads upto the ultimate load, at which stage they may fail by rupture at a critical section. However, if the gross area of the member yields over a major portion of its length before the rupture load is reached, the member may become non-functional due to excessive elongation. Plates and other rolled sections in tension may also fail by block shear of end bolted regions (*see 6.4.1*).

The factored design tension T , in the members shall satisfy the following requirement:

$$T < T_d$$

where

T_d = design strength of the member.

The design strength of a member under axial tension, T_d is the lowest of the design strength due to yielding of gross section, T_{dg} ; rupture strength of critical section, T_{dn} ; and block shear T_{db} , given in 6.2, 6.3 and 6.4, respectively.

6.2 *Design Strength Due to Yielding of Gross Section*

The design strength of members under axial tension, T_{dg} , as governed by yielding of gross section, is given by

$$T_{dg} = A_g f_y / \gamma_{m0}$$

where

f_y = yield stress of the material,

A_g = gross area of cross-section, and

γ_{m0} = partial safety factor for failure in tension by yielding (*see Table 5*).

6.3 *Design Strength Due to Rupture of Critical Section***6.3.1** *Plates*

The design strength in tension of a plate, T_{dn} , as governed by rupture of net cross-sectional area, A_n , at the holes is given by

$$T_{dn} = 0.9 A_n f_u / \gamma_{m1}$$

where

γ_{m1} = partial safety factor for failure at ultimate stress (*see Table 5*),

f_u = ultimate stress of the material, and

A_n = net effective area of the member given by,

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$$A_n = \left[b - nd_h + \sum_i \frac{p_s^2}{4g_i} \right] t$$

where

- b, t = width and thickness of the plate, respectively,
 d_h = diameter of the bolt hole (2 mm in addition to the diameter of the hole, in case the directly punched holes),
 g = gauge length between the bolt holes, as shown in Fig. 5,
 p_s = staggered-pitch length between line of bolt holes, as shown in Fig. 5,
 n = number of bolt holes in the critical section, and
 i = subscript for summation of all the inclined legs.

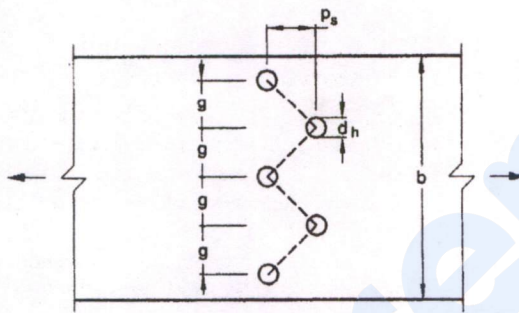


FIG. 5 PLATES WITH BOLTS HOLES IN TENSION

6.3.2 Threaded Rods

The design strength of threaded rods in tension, T_{dn} , as governed by rupture is given by

$$T_{dn} = 0.9 A_n f_u / \gamma_{m1}$$

where

A_n = net root area at the threaded section.

6.3.3 Single Angles

The rupture strength of an angle connected through one leg is affected by shear lag. The design strength, T_{dn} , as governed by rupture at net section is given by:

$$T_{dn} = 0.9 A_{nc} f_u / \gamma_{m1} + \beta A_{go} f_y / \gamma_{m0}$$

where

$$\beta = 1.4 - 0.076 (w/t) (f_u/f_y) (b_s/L_c) \leq \begin{cases} (f_u \gamma_{m0} / f_y \gamma_{m1}) \\ \geq 0.7 \end{cases}$$

where

- w = outstand leg width,
 b_s = shear lag width, as shown in Fig. 6, and

L_c = length of the end connection, that is the distance between the outermost bolts in the end joint measured along the load direction or length of the weld along the load direction.

For preliminary sizing, the rupture strength of net section may be approximately taken as:

$$T_{dn} = \alpha A_n f_u / \gamma_{m1}$$

where

- α = 0.6 for one or two bolts, 0.7 for three bolts and 0.8 for four or more bolts along the length in the end connection or equivalent weld length;
 A_n = net area of the total cross-section;
 A_{nc} = net area of the connected leg;
 A_{go} = gross area of the outstanding leg; and
 t = thickness of the leg.

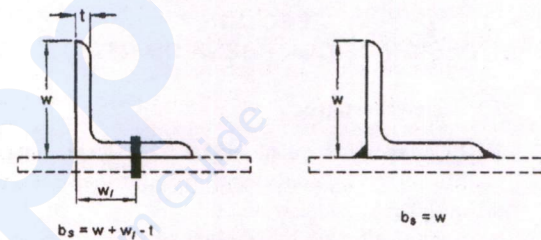


FIG. 6 ANGLES WITH SINGLE LEG CONNECTIONS

6.3.4 Other Section

The rupture strength, T_{dn} , of the double angles, channels, I-sections and other rolled steel sections, connected by one or more elements to an end gusset is also governed by shear lag effects. The design tensile strength of such sections as governed by tearing of net section may also be calculated using equation in 6.3.3, where β is calculated based on the shear lag distance, b_s taken from the farthest edge of the outstanding leg to the nearest bolt/weld line in the connected leg of the cross-section.

6.4 Design Strength Due to Block Shear

The strength as governed by block shear at an end connection of plates and angles is calculated as given in 6.4.1.

6.4.1 Bolted Connections

The block shear strength, T_{db} of connection shall be taken as the smaller of,

$$T_{db} = [A_{vg} f_y / (\sqrt{3} \gamma_{m0}) + 0.9 A_{tn} f_u / \gamma_{m1}]$$

or

$$T_{db} = (0.9 A_{vn} f_u / (\sqrt{3} \gamma_{m1}) + A_{tg} f_y / \gamma_{m0})$$

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where

A_{vg}, A_{vn} = minimum gross and net area in shear along bolt line parallel to external force, respectively (1-2 and 3-4 as shown in Fig. 7A and 1-2 as shown in Fig. 7B),

A_{tg}, A_{tn} = minimum gross and net area in tension from the bolt hole to the toe of the angle, end bolt line, perpendicular to the line of force, respectively (2-3 as shown in Fig. 7B), and

f_u, f_y = ultimate and yield stress of the material, respectively.

6.4.2 Welded Connection

The block shear strength, T_{db} shall be checked for welded end connections by taking an appropriate section in the member around the end weld, which can shear off as a block.

SECTION 7 DESIGN OF COMPRESSION MEMBERS

7.1 Design Strength

7.1.1 Common hot rolled and built-up steel members used for carrying axial compression, usually fail by flexural buckling. The buckling strength of these members is affected by residual stresses, initial bow and accidental eccentricities of load. To account for all these factors, the strength of members subjected to axial compression is defined by buckling class a, b, c, or d as given Table 7.

7.1.2 The design compressive strength P_d , of a member is given by:

$$P < P_d$$

where

$$P_d = A_e f_{cd}$$

where

A_e = effective sectional area as defined in 7.3.2, and

f_{cd} = design compressive stress, obtained as per 7.1.2.1.

7.1.2.1 The design compressive stress, f_{cd} , of axially loaded compression members shall be calculated using the following equation:

$$f_{cd} = \frac{f_y / \gamma_{m0}}{\phi + [\phi^2 - \lambda^2]^{0.5}} = \chi f_y / \gamma_{m0} \leq f_y / \gamma_{m0}$$

where

$$\phi = 0.5 [1 + \alpha (\lambda - 0.2) + \lambda^2]$$

λ = non-dimensional effective slenderness ratio

$$= \sqrt{f_y / f_{cc}} = \sqrt{f_y \left(\frac{KL}{r} \right)^2 / \pi^2 E}$$

$$f_{cc} = \text{Euler buckling stress} = \frac{\pi^2 E}{\left(\frac{KL}{r} \right)^2}$$

where

KL/r = effective slenderness ratio or ratio of effective length, KL to appropriate radius of gyration, r ;

α = imperfection factor given in Table 7;

χ = stress reduction factor (see Table 8) for different buckling class, slenderness ratio and yield stress

$$= \frac{1}{\phi + (\phi^2 - \lambda^2)^{0.5}}$$

λ_{m0} = partial safety factor for material strength.

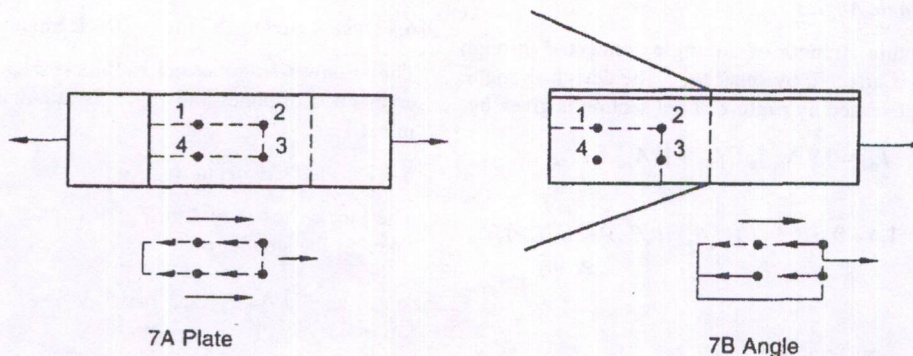
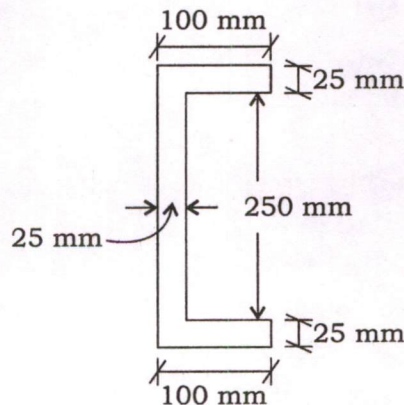


FIG. 7 BLOCK SHEAR FAILURE

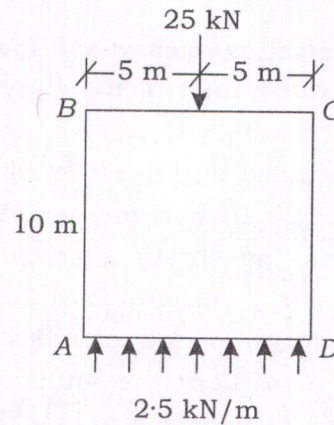
SECTION—B

5. (a) (i) What is smart concrete? Write the key features and benefits of smart concrete. 6
- (ii) What is self-compacting concrete? How is it obtained? Explain the advantage and disadvantage of it. 6
- (b) A vibration test is conducted on the model of a tank. A cable attached to the tank induces a force of 20 kN horizontally and pulls the tank by 5 mm. The cable is cut and the resulting vibration is recorded. At the end of four complete cycles, the time elapsed is 2 seconds and the amplitude is 0.5 mm. Compute the damping ratio, natural period of vibration (undamped), stiffness and damping coefficient. Also find the number of cycles required for the displacement amplitude to decrease to $\left(\frac{1}{10}\right)$ th of the initial amplitude. 12
- (c) Explain the following terms, with sketches, pertaining to the masonry walls :
- (i) Solid wall with piers
- (ii) Cavity wall
- (iii) Faced wall
- (iv) Veneered wall 12
- (d) (i) List the factors that affect strength of a steel column. 4
- (ii) Briefly explain the possible modes of failure of axially loaded steel column. 8
- (e) If the vertical shearing force acting on the thin-walled channel section shown in the figure below is 2000 N, compute and show the shear flow, and determine the shear centre :



12

6. (a) The culvert shown below is of constant section throughout and the top beam is subjected to a central concentrated load of 25 kN. Assuming that the base pressure is uniform throughout, analyze the box culvert. Draw the bending moment diagram using moment distribution method :



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- (b) A rectangular beam $300 \text{ mm} \times 600 \text{ mm}$ is prestressed with parabolic cables having cross-sectional area 1200 mm^2 . The parabolic cables have an eccentricity of 150 mm at mid-span and zero eccentricity at the ends. The beam is simply supported over a span of 12 m and the initial prestress in the cables is 1100 MPa. Estimate the deflection of the beam due to initial prestress plus self-weight of the beam. Assume losses of prestress as 18%. The beam is subjected to a live load of 20 kN/m over its entire span. Estimate the final deflection. Also derive only the expression for deflection due to prestress of the parabolic cable. The unit weight of concrete is 25 kN/m^3 . Assume $E_s = 206 \text{ kN/mm}^2$ and $E_c = 35 \text{ kN/mm}^2$.

20

- (c) (i) An element in a stressed material has tensile stress of 400 MN/m^2 and a compressive stress of 300 MN/m^2 acting on two mutually perpendicular planes and equal shear stresses of 80 MN/m^2 on these planes. Find the principal stresses and position of the principal planes. Find also the maximum shearing stress. Solve using analytical method.

10

- (ii) A simply supported beam 150 mm wide and 220 mm deep is 4 m long and carries a load of 20 kN at mid-span. The load is inclined at an angle of 30° to the vertical. The line of action is passing through the centroid of the section. Find the locations and magnitudes of maximum tensile and compressive stresses set up due to bending.

10

7. (a) (i) What is ferrocement and fiber-reinforced concrete? Write their advantages and disadvantages. 8
- (ii) Describe, in short, the various methods of proportioning concrete. 6
- (iii) Write, in short, the various advantages of RC structures over other masonry structures. 6
- (b) A simply supported reinforced concrete (RC) slab, having a clear span of 3 m, is supported only on two opposite sides on brick walls of 230 mm thick. If the live load on the slab is 3 kN/m^2 and floor finish being 1 kN/m^2 , design the RC slab as per limit state method. Use M 30 concrete and Fe 500 grade steel. Also check for deflection. Sketch the reinforcement details. Assume clear cover to the reinforcement as 20 mm.
- Relevant portion of the IS 456 is enclosed. 20

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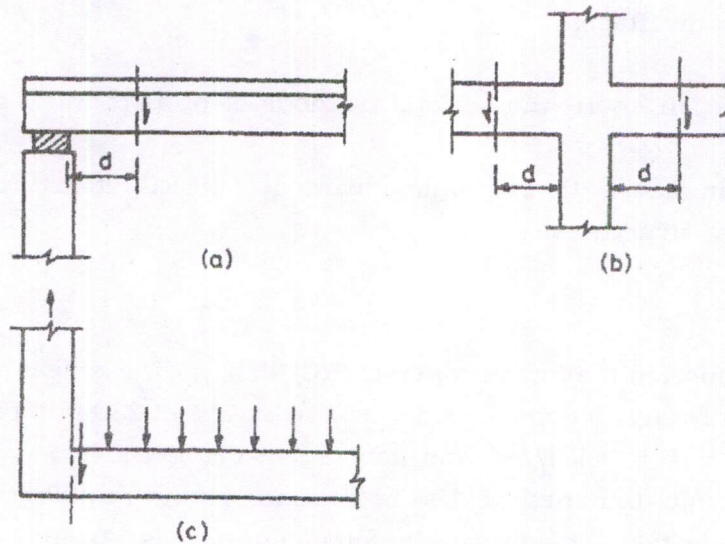


FIG. 2 TYPICAL SUPPORT CONDITIONS FOR LOCATING FACTORED SHEAR FORCE

but in no case greater than the breadth of the web plus half the sum of the clear distances to the adjacent beams on either side.

- a) For T-beams, $b_f = \frac{l_0}{6} + b_w + 6 D_f$
- b) For L-beams, $b_f = \frac{l_0}{12} + b_w + 3 D_f$
- c) For isolated beams, the effective flange width shall be obtained as below but in no case greater than the actual width:

$$\text{T-beam, } b_f = \frac{l_0}{\left(\frac{l_0}{b}\right) + 4} + b_w$$

$$\text{L-beam, } b_f = \frac{0.5 l_0}{\left(\frac{l_0}{b}\right) + 4} + b_w$$

where

- b_f = effective width of flange,
 l_0 = distance between points of zero moments in the beam,
 b_w = breadth of the web,
 D_f = thickness of flange, and
 b = actual width of the flange.

NOTE — For continuous beams and frames, ' l_0 ' may be assumed as 0.7 times the effective span.

23.2 Control of Deflection

The deflection of a structure or part thereof shall not adversely affect the appearance or efficiency of the

structure or finishes or partitions. The deflection shall generally be limited to the following:

- a) The final deflection due to all loads including the effects of temperature, creep and shrinkage and measured from the as-cast level of the supports of floors, roofs and all other horizontal members, should not normally exceed span/250.
- b) The deflection including the effects of temperature, creep and shrinkage occurring after erection of partitions and the application of finishes should not normally exceed span/350 or 20 mm whichever is less.

23.2.1 The vertical deflection limits may generally be assumed to be satisfied provided that the span to depth ratios are not greater than the values obtained as below:

- a) Basic values of span to effective depth ratios for spans up to 10 m:

Cantilever	7
Simply supported	20
Continuous	26

- b) For spans above 10 m, the values in (a) may be multiplied by 10/span in metres, except for cantilever in which case deflection calculations should be made.
- c) Depending on the area and the stress of steel for tension reinforcement, the values in (a) or (b) shall be modified by multiplying with the modification factor obtained as per Fig. 4.
- d) Depending on the area of compression reinforcement, the value of span to depth ratio be further modified by multiplying with the modification factor obtained as per Fig. 5.

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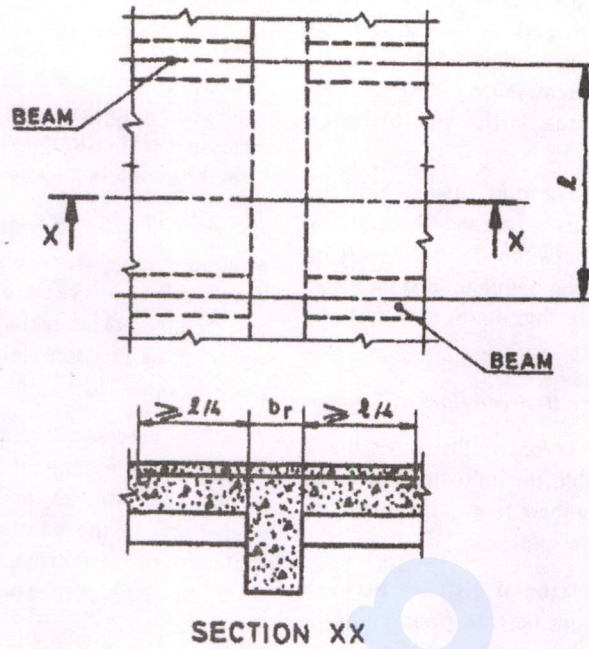


FIG. 3 TRANSVERSE REINFORCEMENT IN FLANGE OF T-BEAM WHEN MAIN REINFORCEMENT OF SLAB IS PARALLEL TO THE BEAM

e) For flanged beams, the values of (a) or (b) be modified as per Fig. 6 and the reinforcement percentage for use in Fig. 4 and 5 should be based

on area of section equal to $b_f d$.

NOTE—When deflections are required to be calculated, the method given in Annex C may be used.

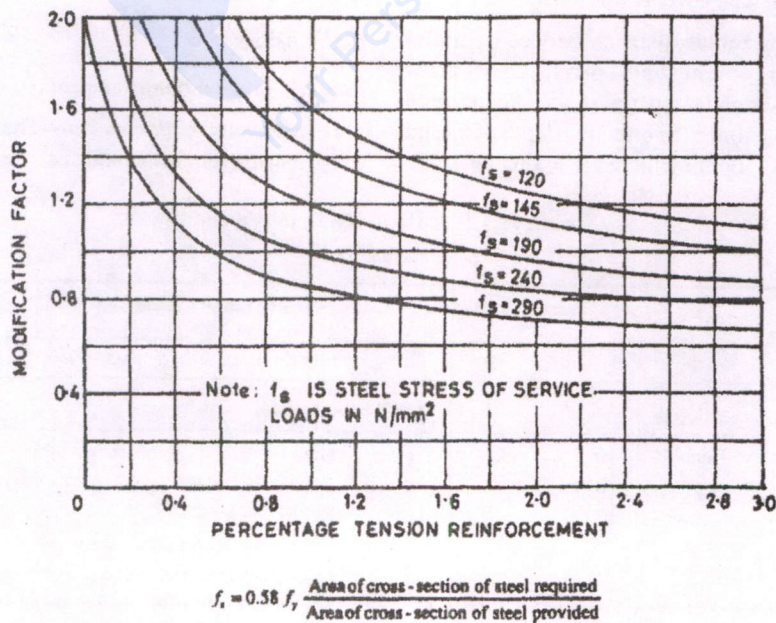


FIG. 4 MODIFICATION FACTOR FOR TENSION REINFORCEMENT

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used the horizontal distance between bars of a group may be reduced to two-thirds the nominal maximum size of the coarse aggregate, provided that sufficient space is left between groups of bars to enable the vibrator to be immersed.

- c) Where there are two or more rows of bars, the bars shall be vertically in line and the minimum vertical distance between the bars shall be 15 mm, two-thirds the nominal maximum size of aggregate or the maximum size of bars, whichever is greater.

26.3.3 Maximum Distance Between Bars in Tension

Unless the calculation of crack widths shows that a greater spacing is acceptable, the following rules shall be applied to flexural members in normal internal or external conditions of exposure.

- a) *Beams* — The horizontal distance between parallel reinforcement bars, or groups, near the tension face of a beam shall not be greater than the value given in Table 15 depending on the amount of redistribution carried out in analysis and the characteristic strength of the reinforcement.
- b) *Slabs*
- 1) The horizontal distance between parallel main reinforcement bars shall not be more than three times the effective depth of solid slab or 300 mm whichever is smaller.
 - 2) The horizontal distance between parallel reinforcement bars provided against shrinkage and temperature shall not be more than five times the effective depth of a solid slab or 450 mm whichever is smaller.

26.4 Nominal Cover to Reinforcement

26.4.1 Nominal Cover

Nominal cover is the design depth of concrete cover to all steel reinforcements, including links. It is the dimension used in design and indicated in the drawings. It shall be not less than the diameter of the bar.

26.4.2 Nominal Cover to Meet Durability Requirement

Minimum values for the nominal cover of normal-weight aggregate concrete which should be provided to all reinforcement, including links depending on the condition of exposure described in 8.2.3 shall be as given in Table 16.

26.4.2.1 However for a longitudinal reinforcing bar in a column nominal cover shall in any case not be less than 40 mm, or less than the diameter of such bar. In the case of columns of minimum dimension of 200 mm or under, whose reinforcing bars do not exceed 12 mm, a nominal cover of 25 mm may be used.

26.4.2.2 For footings minimum cover shall be 50 mm.

26.4.3 Nominal Cover to Meet Specified Period of Fire Resistance

Minimum values of nominal cover of normal-weight aggregate concrete to be provided to all reinforcement including links to meet specified period of fire resistance shall be given in Table 16A.

26.5 Requirements of Reinforcement for Structural Members

26.5.1 Beams

26.5.1.1 Tension reinforcement

- a) *Minimum reinforcement*—The minimum area of tension reinforcement shall be not less than that

Table 15 Clear Distance Between Bars

(Clause 26.3.3)

f_y	Percentage Redistribution to or from Section Considered				
	-30	-15	0	+15	+30
Clear Distance Between Bars					
N/mm ²	mm	mm	mm	mm	mm
250	215	260	300	300	300
415	125	155	180	210	235
500	105	130	150	175	195

NOTE — The spacings given in the table are not applicable to members subjected to particularly aggressive environments unless in the calculation of the moment of resistance, f_y has been limited to 300 N/mm² in limit state design and σ_s limited to 165 N/mm² in working stress design.

ANNEX G

(Clause 38.1)

IS 456 : 2000

MOMENTS OF RESISTANCE FOR RECTANGULAR AND T-SECTIONS

G-0 The moments of resistance of rectangular and T-sections based on the assumptions of 38.1 are given in this annex.

G-1 RECTANGULAR SECTIONS

G-1.1 Sections Without Compression Reinforcement

The moment of resistance of rectangular sections without compression reinforcement should be obtained as follows :

- a) Determine the depth of neutral axis from the following equation :

$$\frac{x_u}{d} = \frac{0.87 f_y A_{st}}{0.36 f_{ck} b d}$$

- b) If the value of x_u/d is less than the limiting value (see Note below 38.1), calculate the moment of resistance by the following expression :

$$M_u = 0.87 f_y A_{st} d \left(1 - \frac{A_{st} f_y}{b d f_{ck}} \right)$$

- c) If the value of x_u/d is equal to the limiting value, the moment of resistance of the section is given by the following expression :

$$M_{u,lim} = 0.36 \frac{x_{u,max}}{d} \left(1 - 0.42 \frac{x_{u,max}}{d} \right) b d^2 f_{ck}$$

- d) If x_u/d is greater than the limiting value, the section should be redesigned.

In the above equations,

- x_u = depth of neutral axis,
- d = effective depth,
- f_y = characteristic strength of reinforcement,
- A_{st} = area of tension reinforcement,
- f_{ck} = characteristic compressive strength of concrete,
- b = width of the compression face,
- $M_{u,lim}$ = limiting moment of resistance of a section without compression reinforcement, and
- $x_{u,max}$ = limiting value of x_u from 39.1.

G-1.2 Section with Compression Reinforcement

Where the ultimate moment of resistance of section

exceeds the limiting value, $M_{u,lim}$ compression reinforcement may be obtained from the following equation :

$$M_u - M_{u,lim} = f_{sc} A_{sc} (d - d')$$

where

$M_u, M_{u,lim}, d$ are same as in G-1.1,

f_{sc} = design stress in compression reinforcement corresponding to a strain of

$$0.0035 \frac{(x_{u,max} - d')}{x_{u,max}}$$

where

$x_{u,max}$ = the limiting value of x_u from 38.1,

A_{sc} = area of compression reinforcement, and

d' = depth of compression reinforcement from compression face.

The total area of tension reinforcement shall be obtained from the following equation :

$$A_{st} = A_{st1} + A_{st2}$$

where

A_{st} = area of the total tensile reinforcement,

A_{st1} = area of the tensile reinforcement for a singly reinforced section for $M_{u,lim}$ and

$$A_{st2} = A_{sc} f_{sc} / 0.87 f_y$$

G-2 FLANGED SECTION

G-2.1 For $x_u < D_f$, the moment of resistance may be calculated from the equation given in G-1.1.

G-2.2 The limiting value of the moment of resistance of the section may be obtained by the following equation when the ratio D_f / d does not exceed 0.2 :

$$M_u = 0.36 \frac{x_{u,max}}{d} \left(1 - 0.42 \frac{x_{u,max}}{d} \right) f_{ck} b_w d^2 + 0.45 f_{ck} (b_f - b_w) D_f \left(d - \frac{D_f}{2} \right)$$

where

$M_u, x_{u,max}, d$ and f_{ck} are same as in G-1.1,

b_f = breadth of the compression face/flange,

b_w = breadth of the web, and

D_f = thickness of the flange.

(c) In a material, the principal stresses are 60 MN/m^2 , 48 MN/m^2 and -36 MN/m^2 . Calculate :

(i) Total strain energy

(ii) Volumetric strain energy

(iii) Shear strain energy

(iv) Factor of safety on total strain energy criterion if the material yields at 120 MN/m^2

Take $E = 200 \text{ GN/m}^2$ and $\frac{1}{m} = 0.3$.

20

8. (a) (i) A three-hinged stiffening girder of suspension bridge of span 130 m is subjected to two point loads of 450 kN and 600 kN at distances 25 m from left support and 40 m from right support respectively. The dip of the cable is 12 m. Determine :

(1) Maximum tension in the cable

(2) Shear force and bending moment values for girder at 40 m from the left support

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(ii) A parabolic arch has a span of 20 m and is supported at different levels such that the crown C is 12 m from left support A and 8 m from right support B. The right support is higher than the left support by 2 m and the crown is higher by 1.5 m with respect to right support. The arch is hinged at the two supports and at the crown. Find the bending moment in the arch at a section Q, 4.5 m from the left support.

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(b) A rectangular cantilever reinforced concrete (RC) beam of $300 \text{ mm} \times 600 \text{ mm}$ cross-section has an effective span of 3 m. It is subjected to a dead load (self-weight plus floor finishes) of 35 kN/m and a live load of 1.5 kN/m at service state. The beam is reinforced with four rebars (reinforcing bars) of 25 mm diameter in tension zone and two rebars of 20 mm diameter in compression zone. Assume effective cover to both tension and compression reinforcement as 50 mm. Use M25 concrete and Fe 500 grade steel. Estimate only the 'initial plus creep' deflection due to permanent loads. Creep coefficient $= 1.6$, $E_c = 5000\sqrt{f_{ck}}$. Flexural strength of concrete $f_{cr} = 0.7\sqrt{f_{ck}}$.

Relevant portion of the IS 456 is enclosed.

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ANNEX C

(Clauses 22.3.2, 23.2.1 and 42.1)

CALCULATION OF DEFLECTION

C-1 TOTAL DEFLECTION

C-1.1 The total deflection shall be taken as the sum of the short-term deflection determined in accordance with C-2 and the long-term deflection, in accordance with C-3 and C-4.

C-2 SHORT-TERM DEFLECTION

C-2.1 The short-term deflection may be calculated by the usual methods for elastic deflections using the short-term modulus of elasticity of concrete, E_c and an effective moment of inertia I_{eff} given by the following equation:

$$I_{eff} = \frac{I_r}{1.2 - \frac{M_r}{M} \frac{z}{d} \left(1 - \frac{x}{d}\right) \frac{b_w}{b}}; \text{ but}$$

$$I_r \leq I_{eff} \leq I_{gr}$$

where

I_r = moment of inertia of the cracked section,

M_r = cracking moment, equal to $\frac{f_{cr} I_{gr}}{y_t}$ where

f_{cr} is the modulus of rupture of concrete, I_{gr} is the moment of inertia of the gross section about the centroidal axis, neglecting the reinforcement, and y_t is the distance from centroidal axis of gross section, neglecting the reinforcement, to extreme fibre in tension,

M = maximum moment under service loads,

z = lever arm,

x = depth of neutral axis,

d = effective depth,

b_w = breadth of web, and

b = breadth of compression face.

For continuous beams, deflection shall be calculated using the values of I_r , I_{gr} and M_r modified by the following equation:

$$X_e = k_1 \left[\frac{X_1 + X_2}{2} \right] + (1 - k_1) X_0$$

where

X_e = modified value of X ,

X_1, X_2 = values of X at the supports,

X_0 = value of X at mid span,

k_1 = coefficient given in Table 25, and

X = value of I_r , I_{gr} or M_r as appropriate.

C-3 DEFLECTION DUE TO SHRINKAGE

C-3.1 The deflection due to shrinkage a_{cs} may be computed from the following equation:

$$a_{cs} = k_3 \Psi_{cs} l^2$$

where

k_3 is a constant depending upon the support conditions,

0.5 for cantilevers,

0.125 for simply supported members,

0.086 for members continuous at one end, and

0.063 for fully continuous members.

Ψ_{cs} is shrinkage curvature equal to $k_4 \frac{\epsilon_{cs}}{D}$

where ϵ_{cs} is the ultimate shrinkage strain of concrete (see 6.2.4),

$$k_4 = 0.72 \times \frac{P_1 - P_c}{\sqrt{P_1}} \leq 1.0 \text{ for } 0.25 \leq P_1 - P_c < 1.0$$

$$= 0.65 \times \frac{P_1 - P_c}{\sqrt{P_1}} \leq 1.0 \text{ for } P_1 - P_c \geq 1.0$$

Table 25 Values of Coefficient, k_1

(Clause C-2.1)

k_1	0.5 or less	0.6	0.7	0.8	0.9	1.0	1.1	1.2	1.3	1.4
k_1	0	0.03	0.08	0.16	0.30	0.50	0.73	0.91	0.97	1.0

NOTE — k_1 is given by

$$k_1 = \frac{M_1 + M_2}{M_{r1} + M_{r2}}$$

where

M_1, M_2 = support moments, and

M_{r1}, M_{r2} = fixed end moments.

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where $P_t = \frac{100 A_{st}}{bd}$ and $P_c = \frac{100 A_{sc}}{bd}$

and D is the total depth of the section, and l is the length of span.

C-4 DEFLECTION DUE TO CREEP

C-4.1 The creep deflection due to permanent loads $a_{cc(perm)}$ may be obtained from the following equation:

$$a_{cc(perm)} = a_{i,cc(perm)} - a_{l(perm)}$$

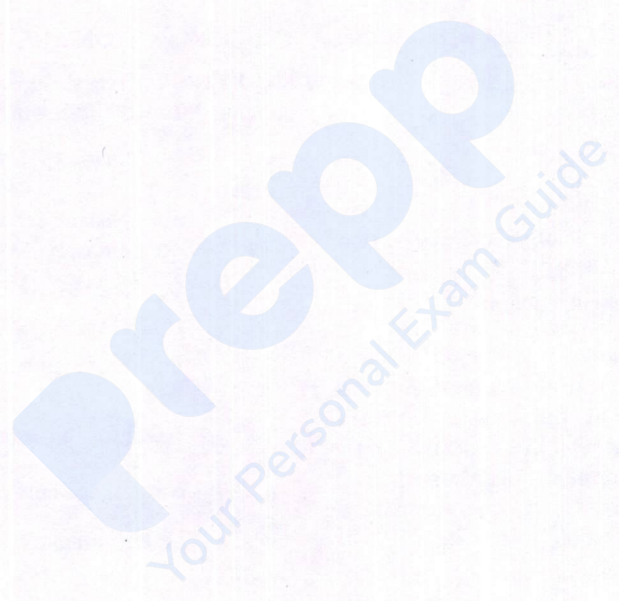
where

$a_{i,cc(perm)}$ = initial plus creep deflection due to permanent loads obtained using an elastic analysis with an effective modulus of elasticity,

$$E_{cc} = \frac{E_c}{1+\theta}; \theta \text{ being the creep coefficient,}$$

and

$a_{l(perm)}$ = short-term deflection due to permanent load using E_c .



(c) (i) Describe the different types of contract in brief. How is a tender document prepared?

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(ii) Estimate the number of carriers, if the data for the project are the following :

Quantity of material to be handled = 5000000 m^3

Capacity of the loaders to be engaged = 2.3 m^3

Capacity of bottom dampers = 30 m^3

Project to be completed in two shifts in 5 years with
yearly working hours = 2000 hr

Job and management factor = 0.70

Operating efficiency = 0.85

Bucket fill factor = 0.85

Swell factor = 0.90

Cycle time for loader = 0.50 minute

Lead distance = 6 km

Speed during empty haul @ 25 km/hr and

loaded haul @ 20 km/hr

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