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ESE - 2025

MAINS EXAMINATION

QUESTIONS WITH DETAILED SOLUTIONS

CIVIL ENGINEERING

(Paper-1)

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CIVIL ENGINEERING

ESE _MAINS_2025_PAPER – I

Questions with Detailed Solutions

SUBJECT WISE WEIGHTAGE

S.No.	NAME OF THE SUBJECT	Marks
1	Strength of Materials	96
2	Structural Analysis	92
3	Building Materials	64
4	Design of Concrete and Masonry Structures	104
5	Steel Structures	52
6	Construction Management & Equipment	72
	Total Marks	**480**

SECTION A

01.(a).(i) Write the qualities of good timber and the factors affecting the strength of timber. [6 M]

Sol: A good quality timber should possess the following characteristics:

i. **Durability:** Resistant to decay, pests, and environmental degradation over time.

ii. **Hardness:** Capable of withstanding wear and mechanical stress without deformation.

iii. **Toughness:** Able to absorb shocks and vibrations without fracturing.

iv. **Elasticity:** Returns to its original shape after removal of load, essential for structural applications.

v. **Workability:** Easy to saw, plane, and shape without splintering or excessive tool wear.

vi. **Uniform Texture and Grain:** Straight grains and even texture enhance strength and aesthetic appeal.

vii. **Low Moisture Content:** Seasoned timber with moisture content below 15% ensures dimensional stability and prevents fungal growth.

viii. **Soundness:** Free from defects such as knots, shakes, splits, and insect damage.

The strength of timber is influenced by several interrelated factors:

i. **Species and Growth Conditions:** Hardwood species (e.g., teak, sal) generally offer higher strength than softwoods; growth rate and soil quality also impact density and grain.

ii. **Moisture Content:** Higher moisture reduces strength; seasoning improves mechanical properties.

iii. **Grain Orientation:** Timber with straight, parallel grains exhibits better tensile and compressive strength.

iv. **Defects:** Presence of knots, shakes, or splits weakens structural integrity.

v. **Age of Tree:** Mature trees (not too young or over-aged) yield stronger timber.

vi. **Preservation and Treatment:** Chemical treatments and proper seasoning enhance durability and load-bearing capacity.

(a) (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 **[6 M]**

Sol:

1. **Brick Buttresses:**

   It is the projecting masonry structures built against a wall to provide lateral support and resist outward thrust. It is commonly used in retaining walls, boundary walls, and tall masonry structures to enhance stability. It can be stepped, sloped, or pilaster-type depending on architectural and structural requirements.

2. **Brick Corbel:**

   It is a series of projecting bricks arranged in steps or courses to support structural loads or decorative elements. It is used to support beams, arches, balconies, or cornices. Each course is projected slightly beyond the one below, maintaining structural balance and avoiding excessive cantilevering.

3. **Brick Coping:**

   It is a protective capping provided at the top of parapet walls or boundary walls. It prevents ingress of rainwater into the wall and enhances durability. Sloped, flat, or weathered coping are some of the types of brick copings which are often finished with mortar, concrete, or stone for improved water resistance.

4. **Thresholds:**

   It is a horizontal members placed at the bottom of door openings which provides a clean transition between rooms, prevents dust and water entry, and supports door frames. It is typically

made of stone, concrete, or durable wood and in brick construction, may be integrated with floor finish.

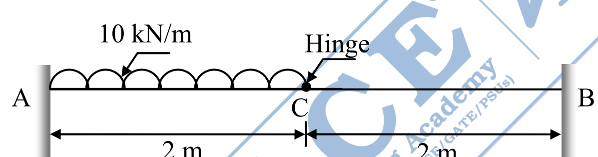
5. **Brick Jambs:**

   The vertical sides of door or window openings constructed using bricks are called Jambs. It supports lintels and frames, and transfers load to adjacent masonry. It may include recesses or projections for fixing frames and should be properly bonded with surrounding wall for strength.

6. **Racking Back:**

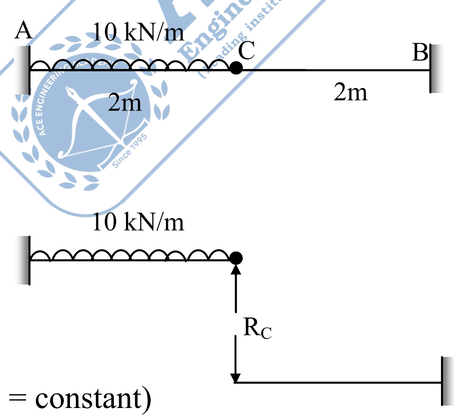
   A stepped arrangement of bricks at the end of a partially completed wall is called Racking Back. Facilitates proper bonding when construction resumes; avoids vertical joints which weaken the wall. Each successive course is set back by half a brick or as per site requirement, ensuring interlocking continuity.

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



[12 M]

Sol:



To calculate R_C (EI = constant)

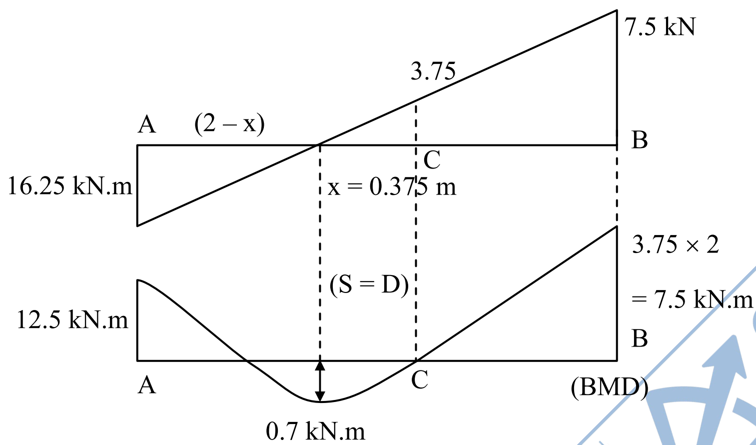
$(y)_{AC@C} = (y)_{BC@C}$

$$\left[\frac{w\ell^4}{8EI} - \frac{R_C\ell^3}{3EI}\right]_{AC} = \left[\frac{R_C\ell^3}{3EI}\right]_{BC}$$

$$\frac{(10)(2)^4}{8} - \frac{R_C(2^3)}{3} = \frac{R_C(2^3)}{3}$$

$$\frac{3\times10\times2^4}{8} = R_C(2^3+2^3)$$

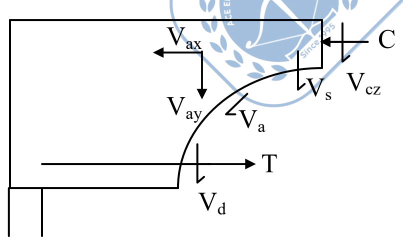
$R_C = 3.75$ kN



(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 M]

Sol: Mechanism of shear transfer in RC beam with shear reinforcement



Internal forces acting at a flexural shear crack

V_{cz} – shear resistance of the uncracked portion of the concrete

V_{ay} – vertical component of the interface shear (aggregate interlock) force

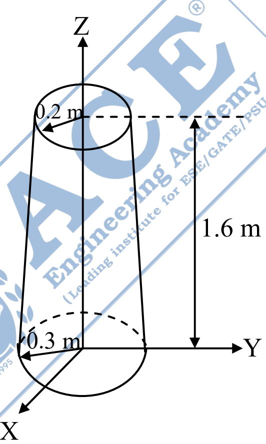
V_d – dowel force in the tension reinforcement (due to dowel action)

V_s – Shear resistance carried by shear reinforcement

$V = V_{cz} + V_{ay} + V_d + V_s$

The relative contribution of the various mechanisms depends on the loading stage, the extent of cracking and the material and the geometric properties of the beam. Prior to flexural cracking, the applied shear is resisted almost entirely by the uncracked concrete ($V \approx V_{cz}$). At the commencement of flexural cracking, there is a redistribution of stresses and some interface shear V_a and dowel action V_d develop. At the stage of diagonal tension cracking, the shear reinforcement that intercepts the crack undergoes a sudden increase in tensile strain and stress. All the four major mechanics are effective at this stage.

(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 M]

Sol: The self weight of frustum

$$= 25\left[\frac{1}{3}\pi\left(0.3^2 - 0.2^2\right)\right]1.6$$

$$w = 2.09 \text{ kN}$$

$$\text{Stress @ base } = \frac{w}{\pi\left(0.3^2\right)} = \frac{2.09}{\pi\left(0.3^2\right)}$$

$$= 7.4 \text{ kN/m}^2$$

(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 M]

Sol: Maximum rimpull of the tractor = maximum tractive effort available - rolling resistance.

Tractor Total Weight: 18,000kg

Weight on Rear Wheels: 60% of 18,000kg = 10,800kg

Coefficient of Traction: 0.65

Maximum Tractive Force at Rear Wheels = 0.65 × 10,800kgf = 7,020kgf

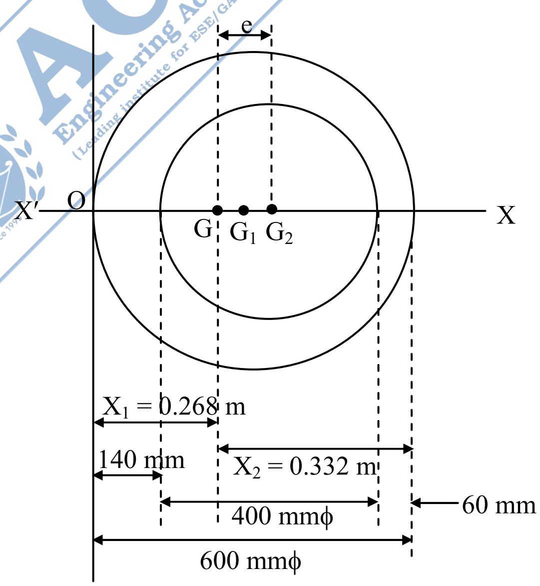
Rolling Resistance: 45kg/ton × 18tons = 810kgf

Maximum Rimpull = Maximum Tractive Force − Rolling Resistance

= 7,020kgf – 810kgf

= 6,210kgf.

02. (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.

[20 M]

Sol: P = 2000 kN

From figure given in question

$$e = 332 - 60 - \frac{400}{2}$$

$$e = 72 \text{ mm}$$

$$I_y = \left[\frac{\pi}{64}(600^4) + \frac{\pi}{4}(600^2)[332-300]^2\right](-)\left[\frac{\pi}{64}(400^4) + \frac{\pi}{4}(400^2)[72^2]\right]$$

$I_y = 4.74 \times 10^9 \text{ mm}^4$

$$A = \frac{\pi}{4}(600^2 - 400^2) = 157 \text{ mm}^2$$

$$\sigma_{max} = \frac{P}{A} + \frac{P.e}{I}(x_2) \quad = \frac{2000\times10^3}{157} + \frac{(2000\times10^3)(72)}{4.74\times10^9}(332)$$

$= 12738.8 + 10.08 = + 12748$ MPa (comp)

$$\sigma_{min} = \frac{P}{A} - \frac{P.e}{I}(x_1)$$

$$= \frac{2000\times10^3}{157} - \frac{(2000\times10^3)(72)}{4.74\times10^9}(268) \quad = 12738.8 - 8.14$$

$= + 12730.65$ MPa (comp)

(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]**

Sol:

i. Autogenous Shrinkage:

Volume reduction in cement paste due to internal chemical reactions (hydration), without moisture exchange with the environment is called Autogenous Shrinkage. It is caused due to self-desiccation from water consumption during hydration. It can lead to microcracking in high-performance concrete.

ii. Bogue Compounds:

Simple chemical compounds present in raw material fuse together in the rotary kiln at around 1500°C to form complex chemical compounds called as Bogue Compounds which are listed as below:

a. C_3S (Tricalcium silicate)

b. C_2S (Dicalcium silicate)

c. C_3A (Tricalcium aluminate)

d. C_4AF (Tetracalcium aluminoferrite)

iii. Casehardening:

Surface hardening of steel while maintaining a tough core is called as Casehardening. It can be done using methods of carburizing, nitriding, flame or induction hardening. It is used for gears, camshafts, and other wear-resistant components.

iv. Elastomers:

These are polymers with elastic properties which returning to original shape after deformation. Some of the examples are Natural rubber, neoprene, silicone. They are used for seals, gaskets, tires, flexible joints.

v. Guniting:

The process of spraying dry mix concrete pneumatically with water added at the nozzle is known as Guniting. It is mainly used in the construction of tunnel linings, slope stabilization, and repair works.

vi. Scoriaceous Aggregate:

These are lightweight aggregate with vesicular (porous) texture, often from volcanic origin. Ex: Scoria. They are used in the preparation of lightweight concrete, insulation layers.

vii. Self-Desiccation:

The internal drying of cement paste due to water consumption during hydration is called as Self-Desiccation. It reduces internal relative humidity, contributing to autogenous shrinkage.

viii. Shingling:

The process of layering or overlapping of materials (often metal sheets or roofing) is called as Shingling.

ix. Puddling:

The process of sealing soil by compacting wet clay to prevent water seepage. It is used for canal linings, reservoirs, ponds.

x. Wet Rot:

It is the fungal decay of timber due to prolonged exposure to moisture. It weakens wood structure, often accompanied by discoloration and softness. It can be prevented by providing proper ventilation, moisture control, and treatment.

(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, γ_{mb} =1.25.

Relevant portion of the code books is enclosed. **[20 M]**

Sol: Design or factored load = 1.5 ×200 = 300 kN

Effective length of the strut = 2.12 m

Thickness of gusset plate = 20 mm

For struts, the slenderness ratio may be assumed in the range of 100 to 120.

Let us assume a slenderness ratio of 110. Corresponding value of design compressive stress from table 9(c) is 94.6 N/mm^2

$$\text{Cross –sectional area required} = \frac{300\times10^3}{94.6} = 3171.24 \text{ mm}^2$$

Angles placed on opposite sides of the gusset plate:

From IS Hand book No. 1, select 2, ISA 110 ×110 × 10 mm

Area provided = 2 × 21.06

= 42.12 cm^2

= 4212 mm^2

Minimum radius of gyration, $r = r_z = 33.6$ mm

$$\frac{K\ell}{r} = \frac{2120}{33.6} = 63.09 < 180$$

Hence, the selected section is ok

For $\frac{KL}{r} = 63.09$ and f_y = 250 MPa, using table 9(c) of the code

$\frac{KL}{r}$		f_{cd}
60	→	168
63.09	→	?

70 $\rightarrow$ 152

$$f_{cd} = 168 - \frac{(168-152)}{(70-60)}(63.09-60)$$

$f_{cd} = 163.056\ \text{N/mm}^2$

The design compressive strength

$$P_d = A_e f_{cd}$$

$$= 4212 \times 163.056$$

$$= 686.79 \times 10^3\ \text{N}$$

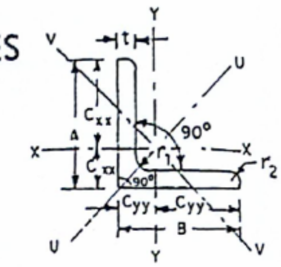
$$= 686.79\ \text{kN} > 300\ \text{kN} \rightarrow \text{It is safe}$$

The designer may now try to economize the section.

TABLE III ROLLED STEEL EQUAL ANGLES

DIMENSIONS AND PROPERTIES

(*Continued*)



Designation	Size $A \times B$ (mm mm)	Thickness t (mm)	Sectional Area a (cm^2)	Weight per Metre w (kg)	Centre of Gravity $C_{xx} = C_{yy}$ (cm)	Distance of Extreme Fibre $e_{xx} = e_{yy}$ (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 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

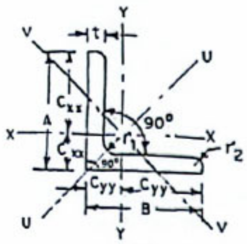


TABLE III ROLLED STEEL EQUAL ANGLES

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	1 174·8	296·0	4·61	5·83	2·93	67·7			435·0	
896·8	1 429·7	363·8	4·58	5·78	2·92	83·5			529·1	
1 048·9	1 668·2	429·5	4·54	5·73	2·91	98·7			616·0	
1 788·9	2 862·0	715·9	6·20	7·84	3·92	122·2	15·0	10·0	1 058·9	ISA 200200
2 197·7	3 511·8	883·7	6·17	7·79	3·91	151·4			1 301·2	
2 588·7	4 130·8	1 046·5	6·13	7·75	3·90	179·9			1 530·5	
3 436·3	5 460·9	1 411·6	6·05	7·63	3·88	243·3			2 015·7	

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:

$$\beta_{lj} = 1.075 - l_j/(200\,d) \text{ but } 0.75 \le \beta_{lj} \le 1.0$$
$$= 1.075 - 0.005(l_j/d)$$

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} = nominal bearing strength of a bolt $= 2.5\,k_b\,d\,t\,f_u$

where

k_b 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:

a) Over size and short slotted holes – 0.7, and
b) 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**.

IS 800 : 2007

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

(Clause 7.1.2.1)

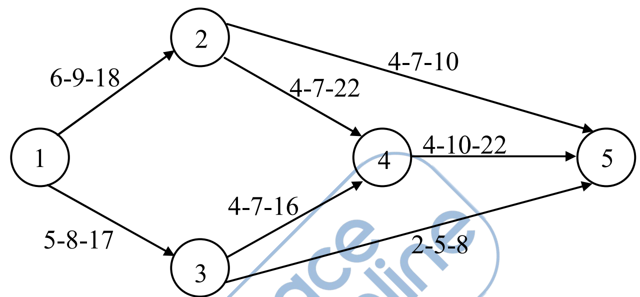
KL/r ↓	Yield Stress, f_y (MPa)																		
	200	210	220	230	240	250	260	280	300	320	340	360	380	400	420	450	480	510	540
10	182	191	200	209	218	227	236	255	273	291	309	327	345	364	382	409	436	464	491
20	182	190	199	207	216	224	233	250	266	283	299	316	332	348	364	388	412	435	458
30	172	180	188	196	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

03.(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

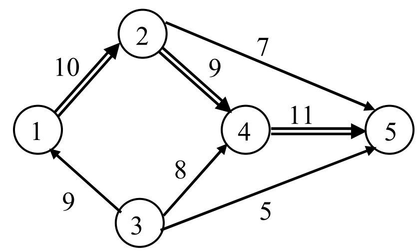


[20 M]

Sol:

Activity	Expected duration $(t_E) = \dfrac{t_o + 4t_m + t_P}{6}$
$1-2$	$\dfrac{6+4(9)+18}{6} = 10$
$1-3$	$\dfrac{5+4(8)+17}{6} = 9$
$2-4$	$\dfrac{4+4(7)+22}{6} = 9$
$3-4$	$\dfrac{4+4(7)+16}{6} = 8$
$2-5$	7
$3-5$	5
$4-5$	$\dfrac{4+4(10)+22}{6} = 11$

Path	**duration**
1 – 2 – 5	17
1 – 2 – 4 – 5	**30**
1 – 3 – 4 – 5	28
1 – 3 – 5	14



Expected project completion time (μ_{CP}) = 30 days

Probability of completing the project in 35 days (X = 35)

$$Z = \frac{X - \mu_{CP}}{\sigma_{CP}} = \frac{35-30}{4.69} = 1.066$$

$$\sigma_{CP} = \sqrt{V_{1-2} + V_{2-4} + V_{4-5}}$$

$$= \sqrt{\left(\frac{18-6}{6}\right)^2 + \left(\frac{22-4}{6}\right)^2 + \left(\frac{22-4}{6}\right)^2} = 4.69$$

$$P(Z) = 84.13 + \frac{0.066 \times 2.3}{0.1} = 85.54\%$$

(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.

[20 M]

Sol: P = 1400 kN, M_x = 60 kNm, M_y = 40 kNm, b = 300 mm, M30, Fe -500, e_c = 60 mm

Factored load and moments

$P_u = 1.5P = 1.5 \times 1400 = 2100$ kN

$M_{ux} = 1.5\ M_x = 1.5 \times 60 = 90$ kNm

$M_{uy} = 1.5\ M_y = 1.5 \times 40 = 60$ kNm

Assume the other side of column is 600 mm

Now size of column is 300 mm × 600 mm

Assume longitudinal steel in percentage

$P \rightarrow 1.5\%$

Check

$$\left(\frac{M_{ux}}{M_{ux_1}}\right)^{\alpha_n} + \left(\frac{M_{uy}}{M_{uy_1}}\right)^{\alpha_n} \le 1$$

$M_{ux} = 90$ kNm

$M_{uy} = 60$ kNm

M_{ux1} = Moment capacity about major axis

M_{uy1} = Moment capacity about minor axis

For M_{ux1}

$$\frac{d'}{D} = \frac{60}{600} = 0.1$$

$$\frac{p}{f_{ck}} = \frac{1.5}{30} = 0.05$$

From chart 48

$$\frac{P_u}{f_{ck}bD} = \frac{2100 \times 10^3}{30 \times 300 \times 600} = 0.38$$

$$\frac{M_{ux_1}}{f_{ck}bD^2} = 0.09$$

$$\frac{M_{ux_1}}{30 \times 300 \times 600^2} = 0.09$$

$M_{ux1} = 291.6 \times 10^6$ Nmm = 291.6 kNm

For M_{uy1}

$$\frac{d'}{D} = \frac{60}{300} = 0.2$$

$$\frac{p}{f_{ck}} = \frac{1.5}{30} = 0.05$$

$$\frac{p_u}{f_{ck}bD} = \frac{2100\times10^3}{30\times600\times300} = 0.38$$

From chart 50

$$\frac{M_{uy_1}}{f_{ck}bD^2} = 0.08$$

$$\frac{M_{uy_1}}{30\times600\times300^2} = 0.08$$

M_{uy1} = 129.6× 10^6 Nmm = 129.6 kNm

From char 63

$$\frac{P_{uz}}{A_g} = 19$$

P_{uz} = 19 × 300 × 600 = 3420 × 10^3 N

$$\frac{P_u}{P_{uz}} = \frac{2100\times10^3}{32420\times10^3} = 0.6$$

$$\alpha_n = 1 + \frac{2-1}{0.8-0.2}\times(0.6-0.2) = 1.67$$

$$\text{Now } \left[\frac{M_{ux}}{M_{ux_1}}\right]^{\alpha} + \left[\frac{M_{uy}}{M_{uy_1}}\right]^{\alpha_n}$$

$$\Rightarrow \left[\frac{90}{291.6}\right]^{1.67} + \left[\frac{60}{129.6}\right]^{1.67}$$

$\Rightarrow 0.42 < 1 \qquad \therefore$ Safe

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:

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

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:

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

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_n is related to P_u/P_{uz}

where $P_{uz} = 0.45 f_{ck} \cdot A_c + 0.75 f_y \cdot A_{sc}$

For values of P_u/P_{uz} = 0.2 to 0.8, the values of α_n vary linearly from 1.0 to 2.0. For values less than 0.2, α_n is 1.0; for values greater than 0.8, α_n 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_{ax} and M_{ay} shall be calculated by the following formulae:

$$M_{ax} = \frac{P_u D}{2\,000}\left\{\frac{l_{ex}}{D}\right\}^2$$

$$M_{ay} = \frac{P_u b}{2\,000}\left\{\frac{l_{ey}}{b}\right\}^2$$

where

P_u = axial load on the member,

l_{ex} = effective length in respect of the major axis,

l_{ey} = effective length in respect of the minor axis,

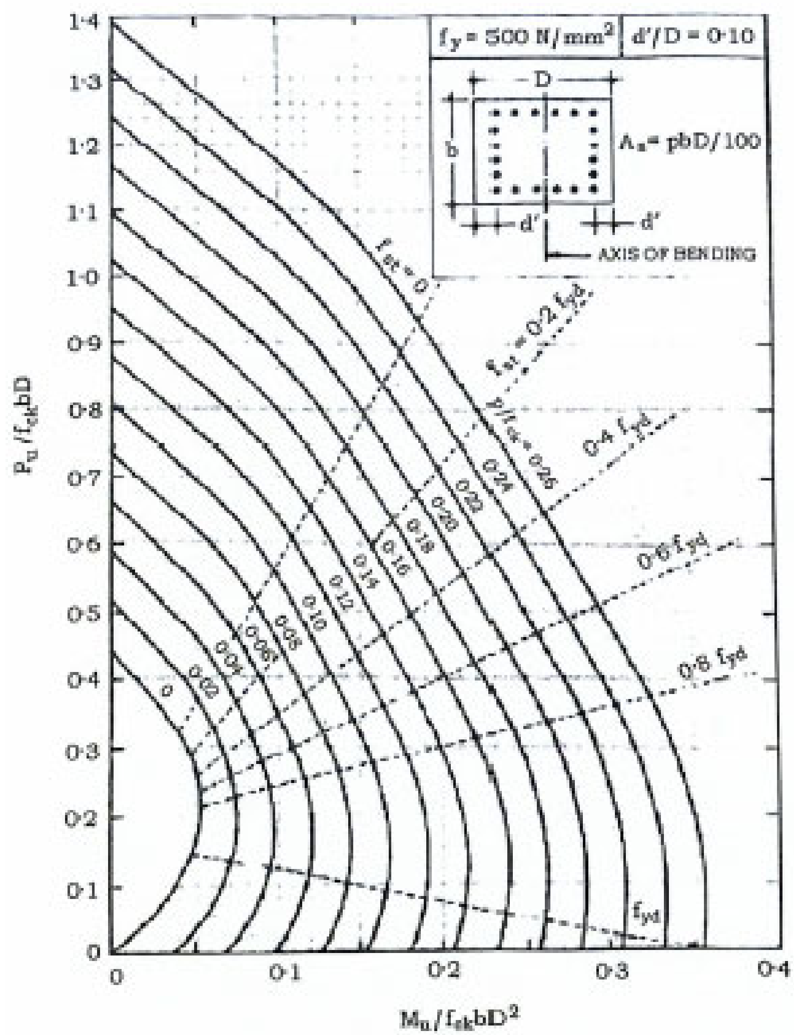
D = depth of the cross-section at right angles to the major axis, and

b = width of the member.

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

f_y
500

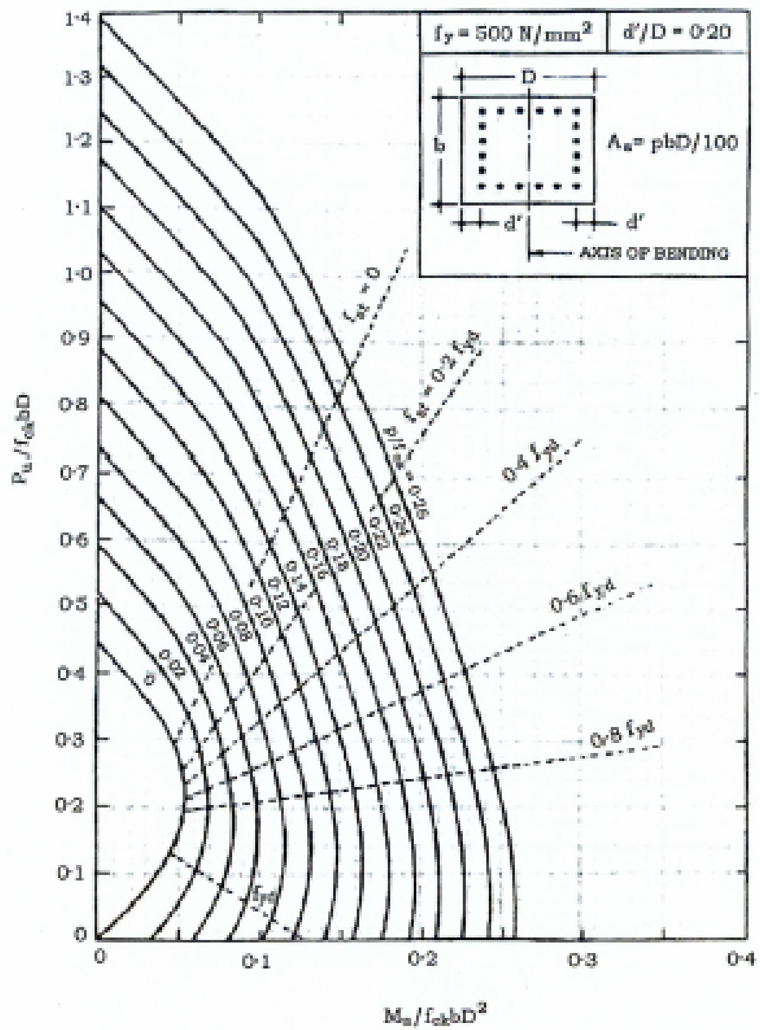
Chart 48 COMPRESSION WITH BENDING—Rectangular Section—Reinforcement Distributed Equally on Four Sides



Compression Members

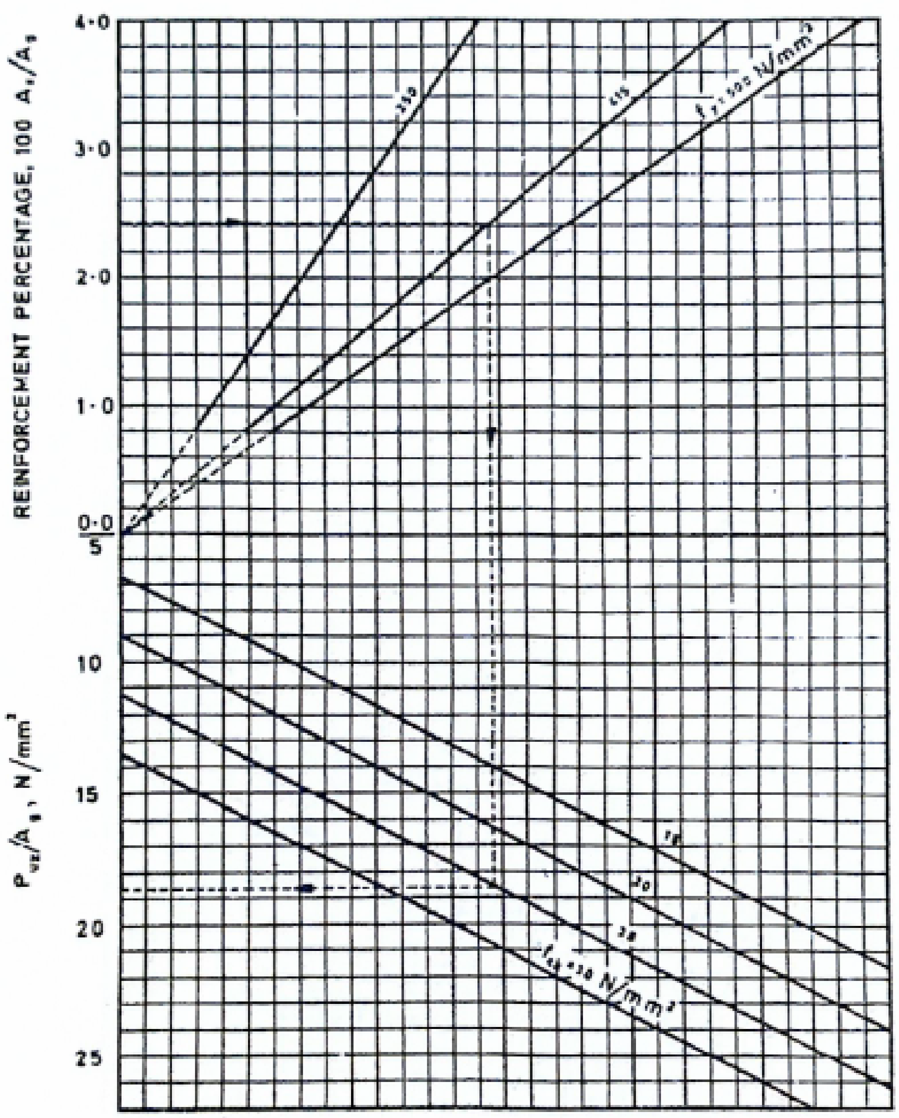
f_y
500

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



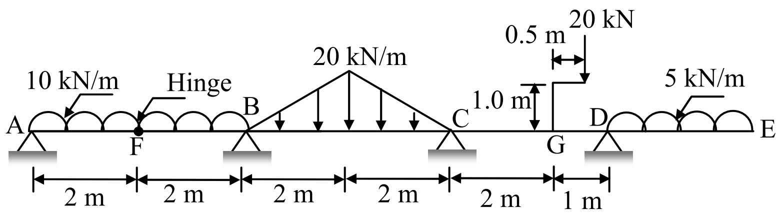
Compression Members

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 GPa, I =10000 cm^4

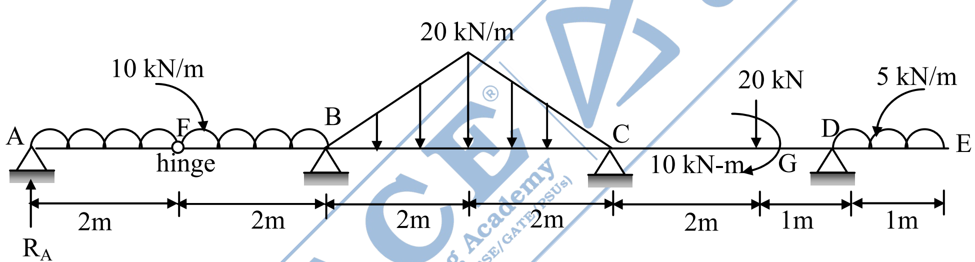


[20 M]

Sol: δ_c =1 cm, 0.01 m

$E = 200\text{ GPa} = 200 \times 10^9\text{ N/m}^2,\ 200 \times 10^6\text{ kN/m}^2$

$I = 10,000\text{ cm}^4 = 10^{-4}\text{ m}^4$



$BM_F = 0$ -ve +ve [left side]

$+R_A \times 2 - 10 \times 2 \times \frac{2}{2} = 0$

$R_A = 10$ kN

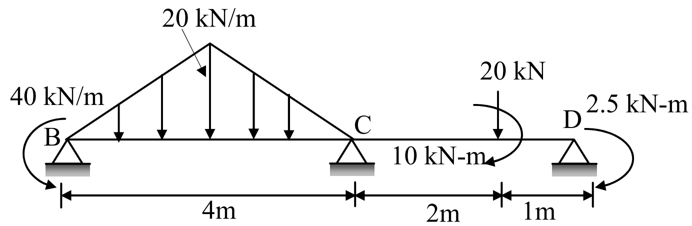
Sagging +ve, hogging –ve

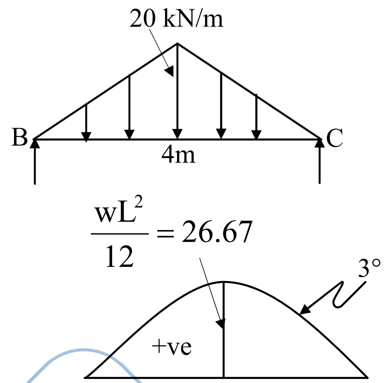
$M_B = 10 \times 4 - 10 \times 4 \times \frac{4}{2} = -40$ kN-m

$M_B = 40$ kN-m (hogging)

$M_D = -5 \times 1 \times \frac{1}{2} = -2.5$ kN-m

$M_D = 2.5$ kN-m (hogging)





BMD

$$\frac{6A_1\bar{x}_1}{L_1} = \frac{5wL_1^3}{32}$$

$$= \frac{5\times 20\times 4^3}{32} = 200$$

$R_C + R_D = 20$

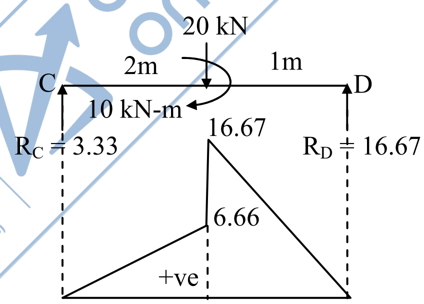
$\sum M_C = 0$ -ve ↺ ↻ +ve

$-R_D \times 3 + 10 + 20 \times 2 = 0$

$R_D = 16.67$ kN

$R_C = 20 - 16.67$

$= 3.33$ kN



$$A_2\bar{x}_2 = \frac{1}{2}\times 1\times 16.67\left[\frac{2}{3}\times 1\right] + \frac{1}{2}\times 2\times 6.66\left[\frac{1}{3}\times 2 + 1\right]$$

$= 16.65$

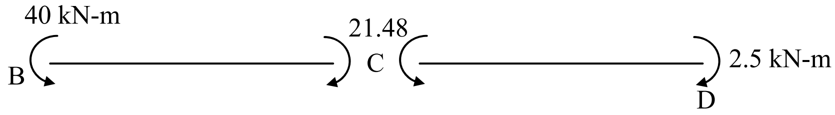
$$\frac{6A_2\bar{x}_2}{L_2} = \frac{6\times 16.65}{3} = 33.3$$

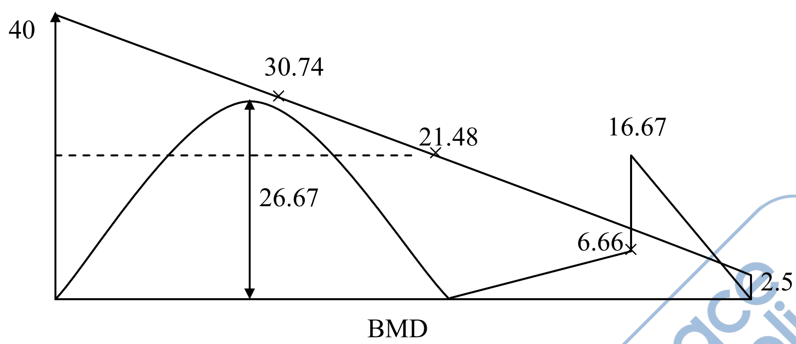
$$M_BL_1 + 2M_C(L_1+L_2) + M_DL_2 = -\overset{200}{\cancel{\frac{6A_1\bar{x}_1}{L_1}}} - \frac{6A_2\bar{x}_2}{L_2} + 6EI\left[\left(\frac{\delta_B-\delta_C}{L_1}\right) + \left(\frac{\delta_C-\delta_D}{L_2}\right)\right]$$

$$40\times 4 + 2M_C[4+3] + 2.5\times 3 = -200 - 33.3 + 6EI\left[-\frac{0.01}{4} + \frac{0.01}{3}\right]$$

$14M_C + 167.5 = -233.3 + 100$

$M_C = -21.48$ kN-m

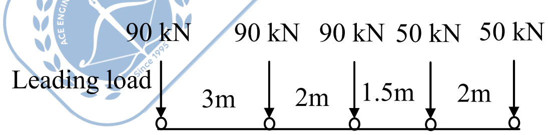


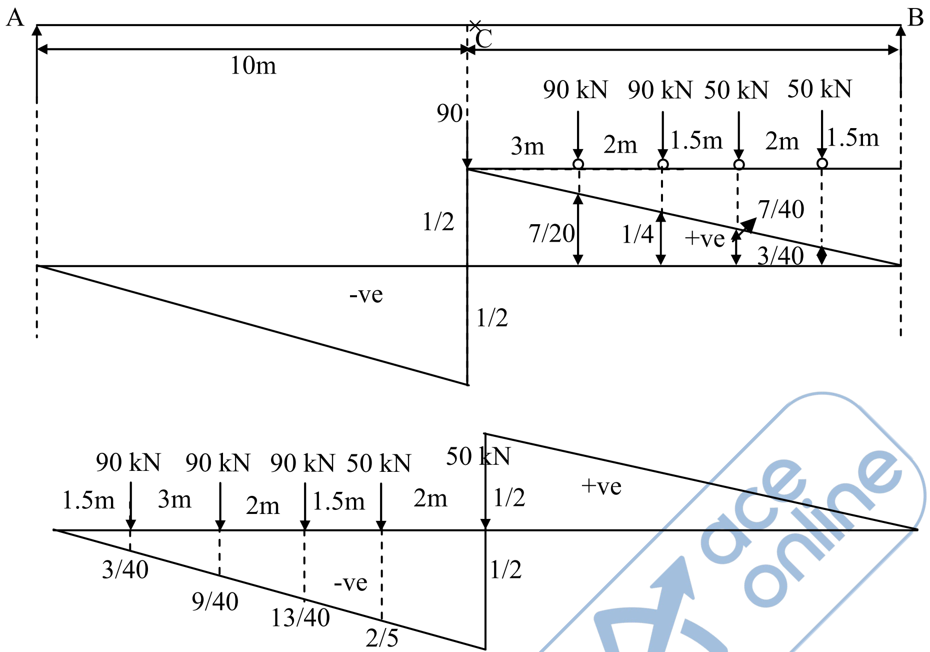


04. (a). Five wheel loads of 90 kN, 90 kN, 90 kN, 50 kN and 50 kN magnitudes spaced 3 m, 2 m, 1.5 m and 2 m apart respectively cross a simply supported girder of 20 m span from right to left with 90 kN load leading. Calculate the maximum positive and negative shear force at the centre of the span and absolute maximum value of bending moment that occurs anywhere in the girder.

[20 M]

Sol:





ILD for SF_C

Maximum +ve SF at centre of span:

SF_C = load × ordinate of ILD under the load

$$= 90\times\frac{1}{2}+90\times\frac{7}{20}+90\times\frac{1}{4}+50\times\frac{7}{40}+50\times\frac{3}{40}$$

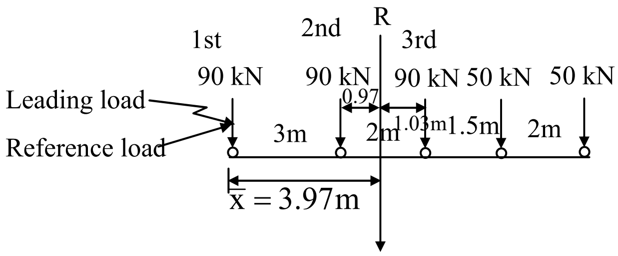
$$= 111.5 \text{ kN}$$

Maximum –ve SF at centre of span:

$$SF_C = -90\times\frac{3}{40}-90\times\frac{9}{40}-90\times\frac{13}{40}-50\times\frac{2}{5}-50\times\frac{1}{2}$$

$$= -101.25 \text{ kN}$$

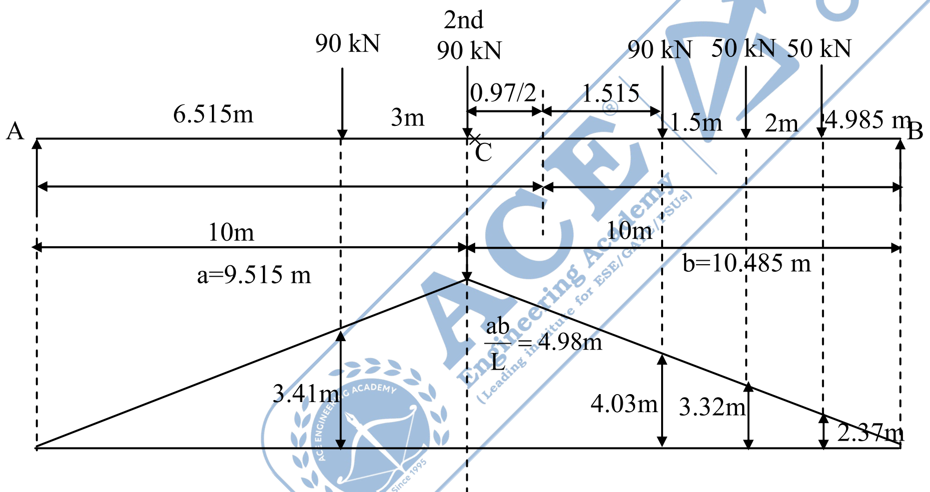
Absolute Maximum Bending Moment:



$$\bar{x} = \frac{90\times 0 + 90\times 3 + 90\times 5 + 50\times 6.5 + 50\times 8.5}{90+90+90+50+50}$$

$\bar{x} = 3.97$ m from reference load

Absolute Maximum Bending occur under 2[nd] 90 kN



Absolute Maximum Bending Moment = 90 × 3.41 + 90 × 4.98 + 90 × 4.03 + 50 × 3.32 + 50 × 2.37

= 1402.3 kN-m

(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 M]

Sol: Initial cost (P) = Rs. 1100

Salvage value (SV) = Rs. 100

Life (n) = 5 years

Rate of interest (i) = 8%

Straight Line Method

Year	**Annual depreciation (D_m)** $D_m = D = \frac{P - SV}{n}$	**Book value (BV_m)** $BV_m = BV_{m-1} - D_m$
0	-	1100
1	200	900
2	200	700
3	200	500
4	200	300
5	200	100

Sum of years (SOY) digit method:

$$SOY = \frac{n(n+1)}{2} = 15$$

$$d_m = \frac{n-(m-1)}{SOY}$$

Year	Annual depreciation (D_m) $D_m = d_m \times (P - SV)$	Book value (BV_m) $BV_m = BV_{m-1} - D_m$
0	-	1100
1	$\frac{5}{15} \times (1100 - 100) = 333.3$	766.66
2	$\frac{4}{15} \times 1000 = 266.66$	500
3	$\frac{3}{15} \times 1000 = 200$	300
4	$\frac{2}{15} \times 1000 = 133.33$	166.66
5	$\frac{1}{15} \times 1000 = 66.66$	100

Declining balance method (single):

$$d = 1 - \left(\frac{SV}{P}\right)^{\frac{1}{n}} = 1 - \left(\frac{100}{1000}\right)^{\frac{1}{5}} = 0.369$$

Year	Annual depreciation (D_m) $D_m = d \times BV_{m-1}$	Book value (BV_m) $BV_m = BV_{m-1} - D_m$
0	-	1100
1	0.369 × 1100 = 405.94	694
2	0.369 × 694 = 256.12	437.87
3	0.369 × 437.89 = 161.57	276.29
4	0.369 × 276.3 = 101.95	174.34
5	0.369 × 174.34 = 64.33	109 ≃ 100

Sinking Fund Factor (SFF) Method:

$$SFF = \frac{i}{(1+i)^n - 1}$$

$$= \frac{0.08}{(1+0.08)^5 - 1}$$

$$= 0.170$$

Annuity (A) = (P – SV) × SFF

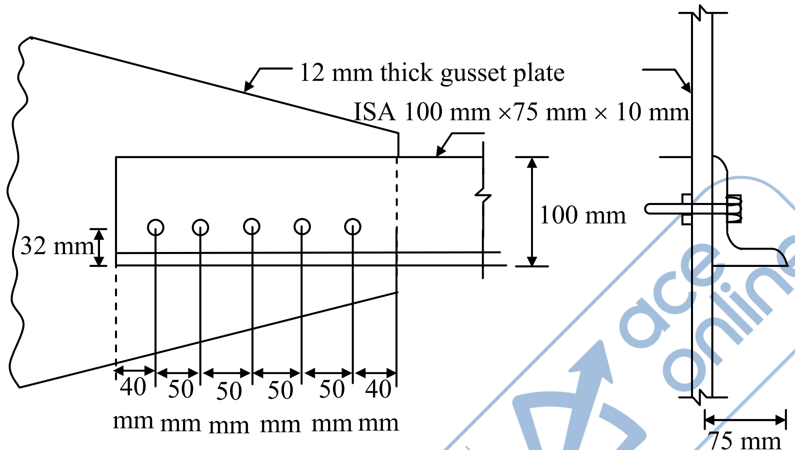
$= (1100 - 100) \times 0.170$

$= 170.45$

Annual depreciation $(D_m) = (1 + i)^{m-1} \times A$

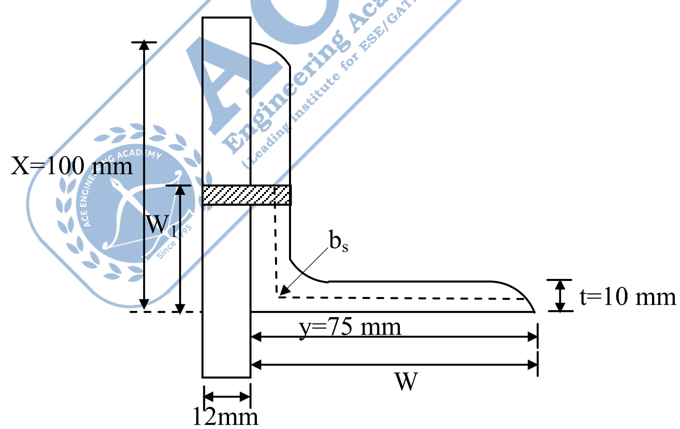
Year	Annuity (A)	Annual depreciation $D_m = (1 + i)^{m-1} \times A$	Book value (BV_m) $BV_m = BV_{m-1} - D_m$
0	-	-	1100
1	170.45	$(1 + 0.08)^{1-1} \times 170.45 = 170.45$	929.55
2	170.45	$(1 + 0.08)^{2-1} \times 170.45 = 184.09$	745.46
3	170.45	$(1 + 0.08)^{3-1} \times 170.45 = 198.81$	546.65
4	170.45	$(1 + 0.08)^{4-1} \times 170.45 = 214.72$	331.93
5	170.45	$(1 + 0.08)^{5-1} \times 170.45 = 231.90$	100

(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 1650mm^2, f_u =410MPa, fy =250MPa, γ_{mo} =1·1, γ_{m1} =1·25. [20 M]



Relevant portion of the IS 800:2007 is enclosed

Sol: Gusset plate is connected to the 100 mm leg of the angle.



d = 16 mm

d_o = 16 + 2 = 18 mm

Net area of connected leg (A_{nc}) = $(x - \frac{t}{2} - d_o)\, t$

$$= \left(100 - \frac{10}{2} - 18\right)10 = 770\ \text{mm}^2$$

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Gross area of outstanding leg (A_{go}) = $\left(y - \frac{t}{2}\right)t$

$$= \left(75 - \frac{10}{2}\right)10 = 700\ \text{mm}^2$$

Gross cross –sectional area of the angle (A_g) = 1650 mm^2

Strength governed by yielding of gross section

$$T_{dg} = \frac{A_g f_y}{\gamma_{mo}} = \frac{1650 \times 250}{1.10} = 375 \times 10^3\ \text{N} = 375\ \text{kN}$$

Strength governed by rupture of critical section

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

$$\beta = 1.4 - 0.076 \frac{W}{t} \times \frac{f_y}{f_u} \times \frac{b_s}{L_c}$$

W = 75 mm

$$b_s = W + \overset{g}{\cancel{W}}_1 - t$$

$$= 75 + 32 - 10 = 97\ \text{mm}$$

L_c = 50 + 50 + 50+50

L_c = 200 mm

$$\beta = 1.4 - 0.076 \times \frac{75}{10} \times \frac{250}{410} \times \frac{97}{200} = 1.23$$

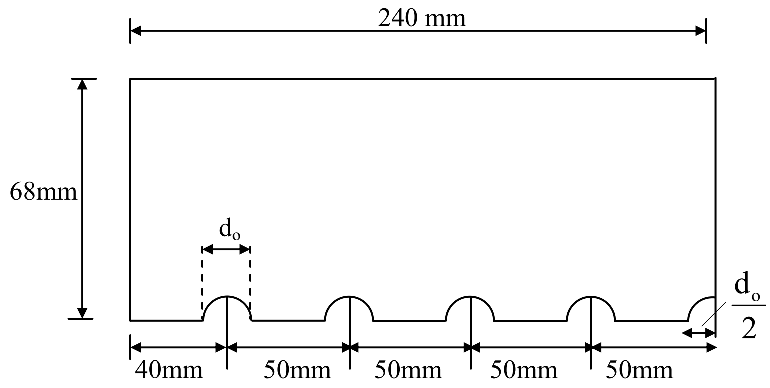
$$0.7 \le \beta \le 0.9 \frac{f_u}{f_y} \times \frac{\gamma_{m0}}{\gamma_{m1}}$$

$$0.7 \le \beta \le 1.44$$

$$T_{dn} = \frac{0.9 \times 770 \times 410}{1.25} + 1.23 \times \frac{700 \times 250}{1.10}$$

$$= 421.39 \times 10^3\ \text{N} = 421.39\ \text{kN}$$

Strength governed by block shear (T_{db})



Gross shear plane area $(A_{vg}) = 240 \times 10 = 2400\ mm^2$

Net shear plane area $(A_{vn}) = (240 - 4 \times d_o - 1 \times \frac{d_o}{2})\ 10$

$$= \left(240 - 4 \times 18 - \frac{18}{2}\right) 10 = 1590\ mm^2$$

Gross tensile plane area $(A_{tg}) = 68 \times 10 = 680\ mm^2$

Net tensile plane area $(A_{tn}) = (68 - 1 \times \frac{d_o}{2})\ 10 = 590\ mm^2$

If shear yield and tension rupture

$$T_{db1} = \frac{A_{vg} f_y}{\sqrt{3}\gamma_{mo}} + \frac{0.9 A_{tn} f_u}{\gamma_{m1}}$$

$$= \frac{2400 \times 250}{\sqrt{3} \times 1.10} + \frac{0.9 \times 590 \times 410}{1.25} = 489.086 \times 10^3\ N = 489.086\ kN$$

If shear rupture and tension yield

$$T_{db2} = \frac{0.9 A_{vn} f_u}{\sqrt{3}\gamma_{m1}} + \frac{A_{tg} f_y}{\gamma_{mo}}$$

$$= \frac{0.9 \times 1590 \times 410}{\sqrt{3} \times 1.25} + \frac{680 \times 250}{1.10} = 425.53 \times 10^3\ N = 425.53\ kN$$

Hence,

$$T_{db} = 375\ kN$$

IS 800 : 2007

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).

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

5.6.3 *Durability*

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

a) Environment,
b) Degree of exposure,
c) Shape of the member and the structural detail,
d) Protective measure, and
e) 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.

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,

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

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.

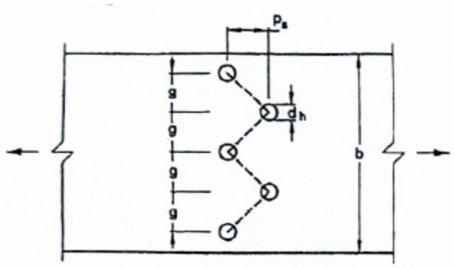


Fig. 5 Plates with Bolts Holes in Tension

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

$$\beta = 1.4 - 0.076\,(w/t)\,(f_y/f_u)\,(b_s/L_c) \le (f_u \gamma_{m0} / f_y \gamma_{m1}) \\ \ge 0.7$$

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.

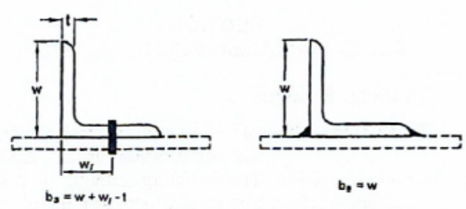


Fig. 6 Angles with Single Leg Connections

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.

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:

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

where

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

λ = non-dimensional effective slenderness ratio

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

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

$$P_d = A_e f_{cd}$$

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

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

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

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

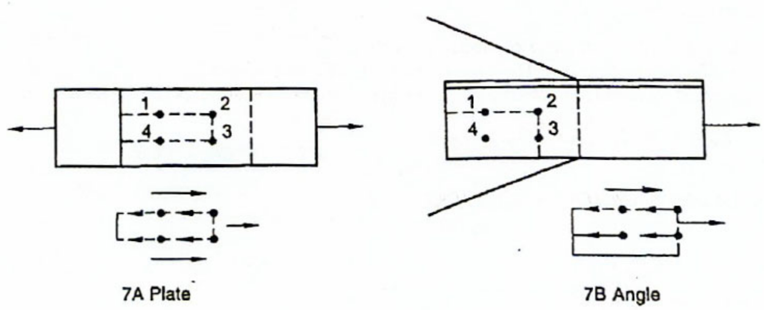

FIG. 7 BLOCK SHEAR FAILURE

SECTION-B

05. (a) (i) What is smart concrete? Write the key features and benefits of smart concrete. [6M]

Sol: Smart concrete is a technologically enhanced form of conventional concrete that incorporates sensors or conductive materials to enable real-time monitoring of structural health and environmental conditions.

Typically, it contains embedded sensors or additives such as carbon fibers, which allow the concrete to detect changes in stress, strain, temperature, and the presence of cracks. This self-sensing capability makes smart concrete particularly valuable in critical infrastructure applications like bridges, tunnels, and high-rise buildings, where early detection of damage can prevent catastrophic failure.

The use of smart concrete improves safety, reduces maintenance costs through predictive diagnostics, and extends the service life of structures. By integrating monitoring functions directly into the material, smart concrete represents a significant advancement in sustainable and intelligent construction practices.

(a). (ii) What is self-compacting concrete? How is it obtained? Explain the advantage and disadvantage of it. [6M]

Sol: Self-compacting concrete (SCC) is a highly flowable and non-segregating concrete that can spread into place and fill formwork under its own weight, without the need for mechanical vibration.

It is achieved through a carefully designed mix that includes high-range water-reducing admixtures (superplasticizers), viscosity-modifying agents, and well-graded aggregates. These components work together to ensure high fluidity while maintaining cohesion and preventing segregation.

SCC offers several advantages, such as faster placement, reduced labour requirements, and superior surface finish, making it ideal for use in structures with dense reinforcement or complex formwork. However, it also has certain disadvantages, including higher material costs due to the use of specialized admixtures and the need for precise mix control to avoid issues like bleeding or loss of stability. Despite these challenges, SCC is widely adopted in modern construction for its efficiency and quality benefits.

(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 (1/10)th of the initial amplitude. [12M]

Sol: Given static force F = 20 kN

Static deflection x_o = 5 mm

Amplified after '4' cycles x_4 = 0.5 mm,

Time taken for 4 cycles = 2 sec

(a) Stiffness k

$$F = Kx_o \Rightarrow K = \frac{F}{x_o} = \frac{20\times10^3}{5\times10^{-3}} = 4\times10^6\ \text{N/m}$$

(b) logarithmic decrement

$$\delta = \frac{1}{n}\ell n\left[\frac{x_o}{x_n}\right] = \frac{1}{4}\ell n\left[\frac{5}{0.5}\right] = 0.57$$

$$\text{Damping ratio}\ \ \varepsilon = \frac{\delta}{\sqrt{4\pi^2+\delta^2}} = \frac{0.57}{\sqrt{4\pi^2+0.57^2}} = 0.09$$

(c) Undamped time period T_n = ?

$$\text{Damped time period}\ T_d = \frac{2\pi}{w_d} = \frac{2\pi}{\sqrt{1-\varepsilon^2}\,w_n}$$

$$= \frac{T_n}{\sqrt{1-\varepsilon^2}} = \frac{2}{4} = 0.5$$

$$T_n = \sqrt{1-\varepsilon^2}\times T_d$$

$$= \sqrt{1-0.09^2}\times 0.5$$

$$= 0.4979\ \text{sec}$$

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Undamped Natural frequency

$$w_n = \frac{2\pi}{T_n} = \frac{2\pi}{0.4979} = 12.61 \text{ rad/s}$$

(d) Mass of the system

$$w_n = \sqrt{\frac{K}{m}} \rightarrow m = \frac{K}{wn^2} = \frac{4\times10^6}{12.61^2} = 2.51 \times 10^4 \text{ kg}$$

(e) damping coefficient 'c' = ?

Damping factor $\varepsilon = \dfrac{C}{2\sqrt{Km}}$

$$C = 2\varepsilon\sqrt{Km}$$

$$= 2\times0.09\times\sqrt{4\times10^6\times2.51\times10^4} = 5.7\times10^4 \text{ N-s/m}$$

(f) if $x_n = \dfrac{x_o}{10}$ $\Rightarrow$ find number of cycles 'n'

$$\delta = \frac{1}{n}\ell n\left[\frac{x_o}{x_n}\right] = \frac{1}{n}\ell n[10] = 0.57$$

$$n = \frac{\ell n10}{0.57} \approx 4 \text{ cycles}$$

(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 **[12M]**

Sol: **(i) Solid wall with piers:** A solid wall is a single leaf wall constructed from a single material like brick (or) stone when combined with the piers, which are projecting columns (or) sections of wall that provide additional support and stability, if forms a solid wall with enhanced structural integrity and load bearing capacity.

(ii) Cavity wall : Cavity wall consists of two separate wall (or) leaves (usually brick (or) block work) with a gap (or) cavity in between them. This cavity provides insulation against heat of loss and sound, and also act as barriers against moisture penetration. The two leaves are typically tied together with wall ties.

(iii) Faced wall: A faced wall is wall where the outer surface is made of different material than the main structural wall behind it. The facing material often of higher quality (or) aesthetic appear (ex: decorative brick, stone (or) precast concrete) is bonded to the backing wall to provide a desired finish and protection.

(iv) veneered wall: It is similar to the faced wall, a veneered wall involves applying a thin layer of material (the veneer) over a structural backing wall. This veneer, which can be brick, stone (or) other decorative materials, is primarily for aesthetic purpose and is not typically load bearing. It is attached to the backing wall using mechanical fasteners (or) mortar.

(d) (i). List the factors that affect strength of a steel column. [4M]

Sol: The strength of a column depends on the following parameters:

1. Material of the column
2. Cross-sectional configuration
3. Length of the column
4. Support conditions at the ends (called restraint conditions)
5. Residual stresses
6. Imperfections

The imperfections include the following:

1. The material not being isotropic and homogenous
2. Geometric variations of columns
3. Eccentricity of load

(d)(ii) Briefly explain the possible modes of failure of axially loaded steel column. **[8M]**

Sol: Modes Possible Failure:

The possible failure modes of an axially loaded column are discussed as follows:

Local buckling Failure occurs by buckling of one or more individual plate elements, e.g., flange or web, with no overall deflection in the direction normal to the applied load. This failure mode may be prevented by selecting suitable width- to-thickness ratios of component plates.

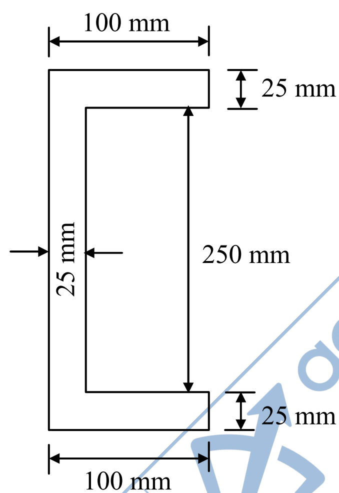
Squashing When the length is relatively small (stocky column) and its component plate elements are prevented from local buckling, then the column will be able to attain its full strength or 'squash load" (yield stress × area of cross section).

Overall flexural buckling This mode of failure normally controls the design of most compression members. In this mode, failure of the member occurs by excessive deflection in the plane of the weaker principal axis. An increase in the length of the column, results in the column resisting progressively less loads

Torsional and flexural-torsional buckling Torsional buck ling failure occurs by wisting about the shear centre in the longitudinal axis. A combination of flexure and twisting, called flexural-torsional buckling is also possible. Torsional buckling is a possible mode of failure for point symmetric sections. Flexural torsional buckling must be checked for open sections that are singly symmetric and for sections that have no symmetry. Note that open sections that are doubly symmetric or point symmetric are not subjected to flexural- torsional buckling, since their shear centre and centroid co-incide. Closed sections are also immune to flexural-torsional buckling

(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 :

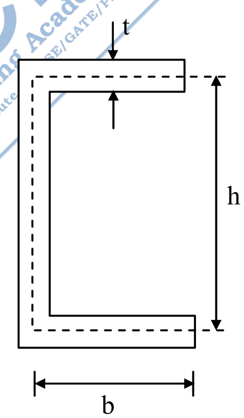
[12M]



Sol: shear centre:

$$e = \frac{b^2h^2t}{4I}$$

Centre line dimensions;



$$b = 100 - \frac{25}{2} = 87.5 \text{ mm}$$

$$h = 250 + 25 = 275 \text{ mm}$$

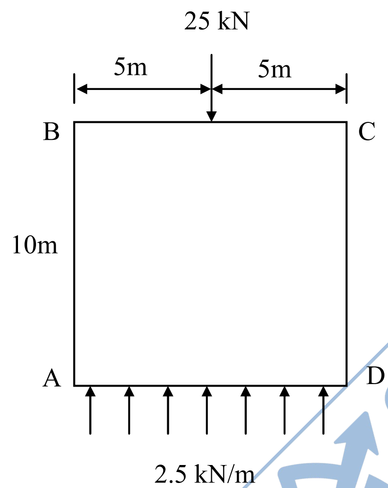
$$I = \frac{100\times300^3}{12} - \frac{75\times250^3}{12}$$

$$= 0.127 \times 10^9 \text{ mm}^4$$

$$\therefore \quad e = \frac{87.5^2\times275^2\times25}{4\left(0.125\times10^9\right)}$$

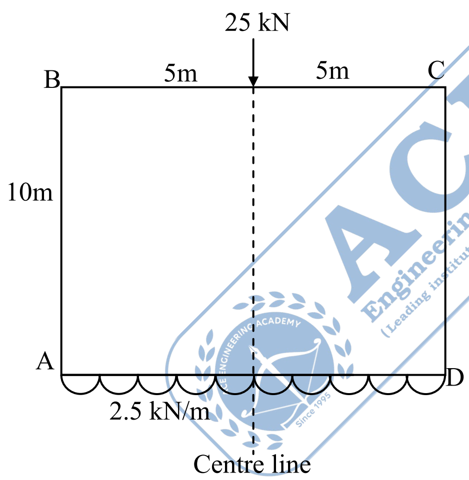
e = 28.9 mm (to the left of centre line of web)

06.(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

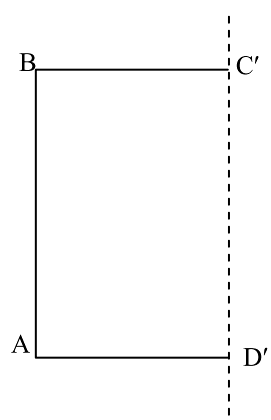


[20M]

Sol:



Using short cut Method of MDM:



Distribution factor (DF):

$$K_{AB} = \frac{4EI}{10} = 0.4EI$$

$$K_{AD'} = \frac{1}{2} \times \frac{4EI}{10} = 0.2EI$$

$$\Sigma K_A = K_{AB} + K_{AD'} = 0.6EI$$

$$K_{BA} = \frac{4EI}{10} = 0.4EI$$

$$\Sigma K_B = K_{BA} + K_{BC'} = 0.6EI$$

$$K_{BC'} = \frac{1}{2} \times \frac{4EI}{10} = 0.2EI$$

$$DF_{AB} = \frac{K_{AB}}{\Sigma K_A} = \frac{0.4EI}{0.6EI} = \frac{2}{3}$$

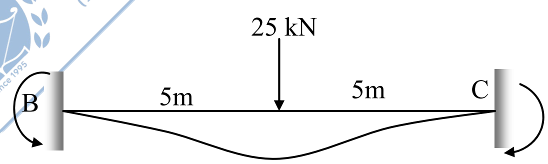
$$DF_{AD'} = \frac{K_{AD'}}{\Sigma K_A} = \frac{0.2EI}{0.6EI} = \frac{1}{3}$$

$$DF_{BA} = \frac{K_{BA}}{\Sigma K_B} = \frac{0.4EI}{0.6EI} = \frac{2}{3}$$

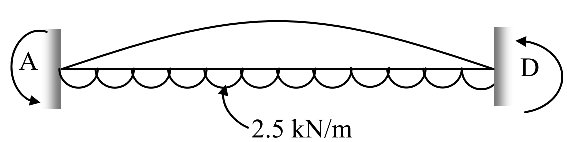
$$DF_{BC'} = \frac{K_{BC'}}{\Sigma K_B} = \frac{0.2EI}{0.6EI} = \frac{1}{3}$$

Fixed end moment (FEM):

$M_{FAB} = M_{FBA} = 0$ [no load]



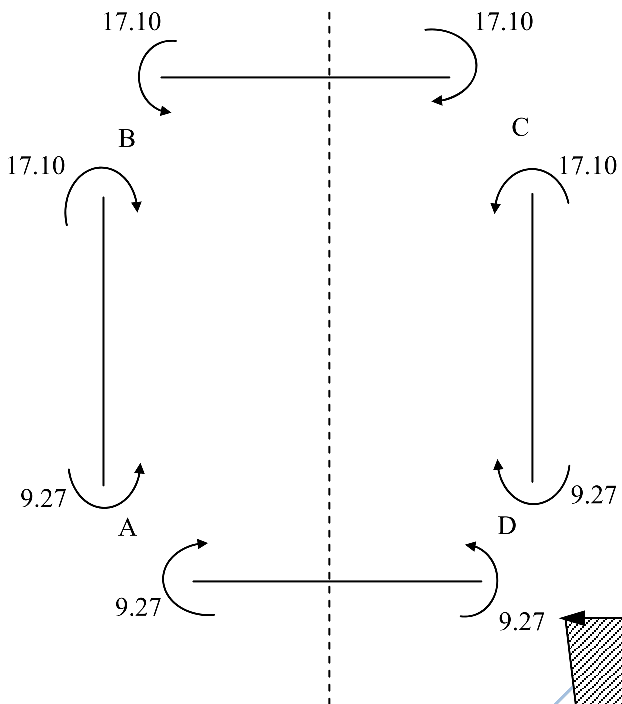
$$M_{FBC} = -\frac{WL}{8} = \frac{-25 \times 10}{8} = -31.25\,kN-m$$

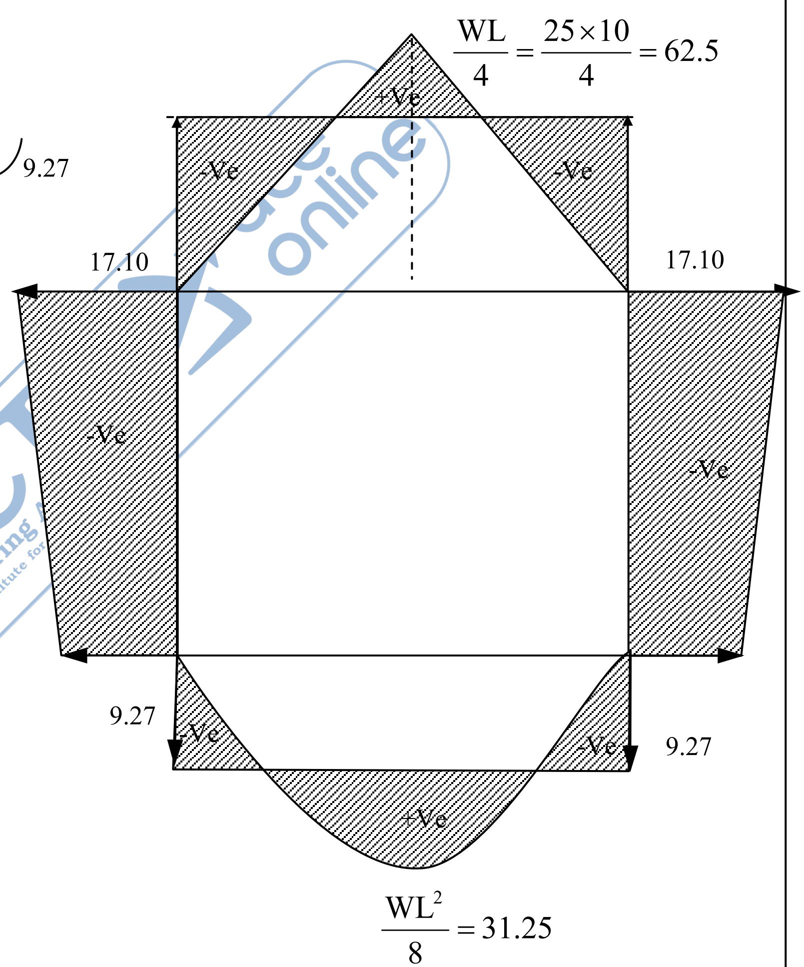


$$M_{FAD} = +\frac{wL^2}{12} = \frac{2.5\times10^2}{12} = 20.83\,\text{kN}-\text{m}$$

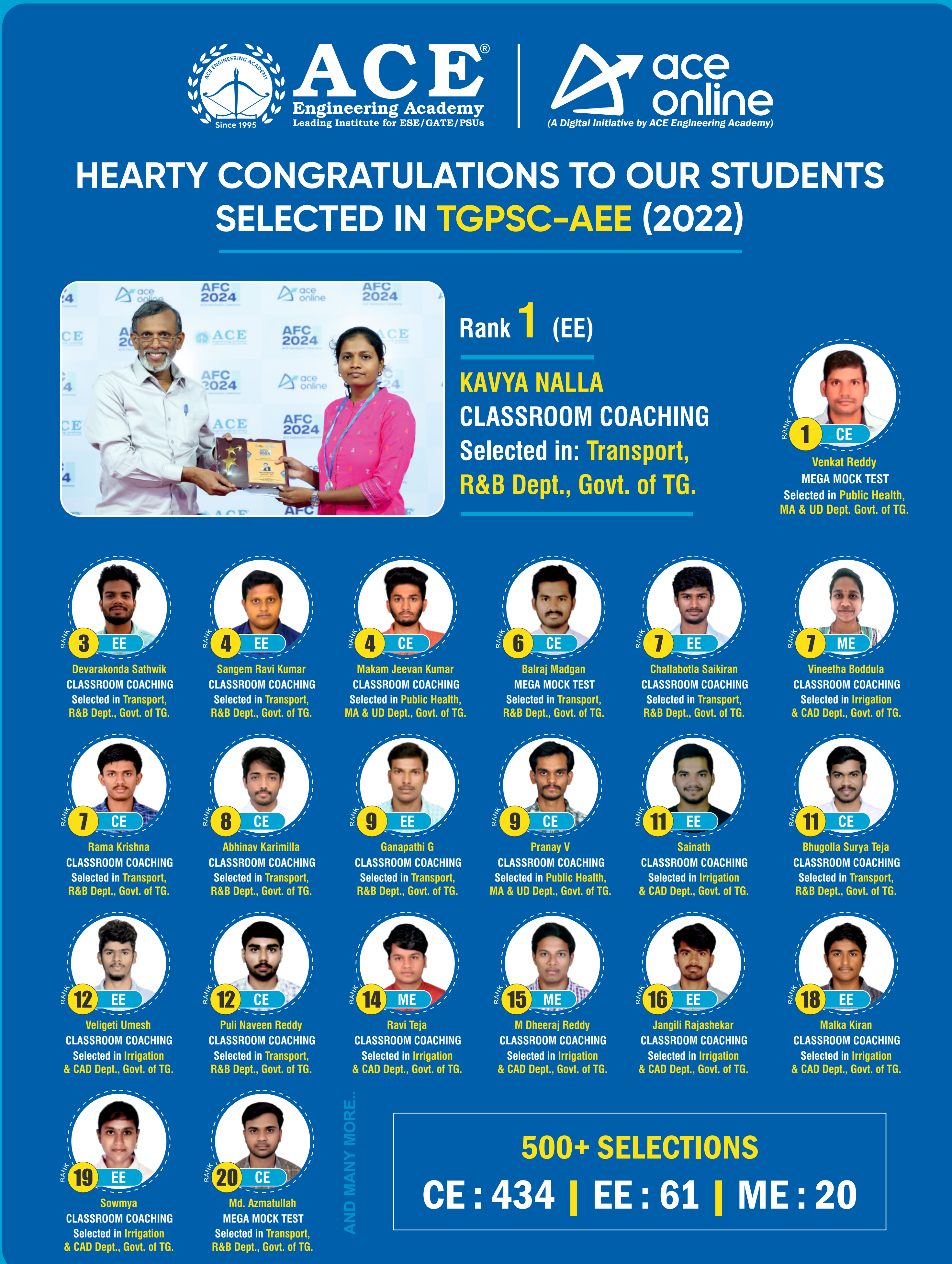
Moment Distribution:

	D′ – A	A – B	B – A	B – C′
Joint	A	A	B	B
Distribution factor	1/3	2/3	2/3	1/3
	20.83	0	0	–31.25
Balance at A & B	–6.94	–13.88	20.83	10.41
Carry over		10.41	–6.94	
Balance at A & B	–3.47	–6.94	4.62	2.31
Carry over		2.31	–3.47	
Balance at A & B	–0.77	–1.54	2.31	1.15
Carry over		1.15	–0.77	
Balance at A & B	–0.38	–0.76	0.51	0.25
Carry over		0.25	–0.38	
Final Moments	9.27	–9.27	+17.10	–17.10


17.10
17.10
B
C
17.10
17.10
9.27
9.27
A
D
9.27
9.27


$\frac{WL}{4} = \frac{25\times10}{4} = 62.5$
+Ve
-Ve
-Ve
17.10
17.10
-Ve
-Ve
9.27
9.27
-Ve
-Ve
+Ve
$\frac{WL^2}{8} = 31.25$

BM diagram



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AND MANY MORE...
500+ SELECTIONS
CE : 434 | EE : 61 | ME : 20

(b). A rectangular beam 300 mm × 600 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 kN/mm^2 and E_c = 35 kN/mm^2.

[20M]

Sol: Given

b =300 mm, D = 600 mm, A_s = 1200 mm^2, e = 150 mm, L = 12 m

σ_o = 1100 MPa, Loss = 18%, LL = 20 kN/m, γ = 25 kN/m^3, E_s = 206 kN/mm^2, E_c = 35 kN/mm^2

Self weight of the beam, $w_D = \gamma bD$

$= 25 \times 0.3 \times 0.6$

$= 4.5$ kN/m

Prestressing force $P_o = \sigma_o A_s$

$= 1100 \times 1200$

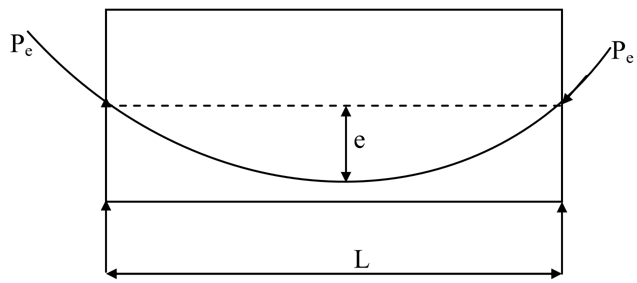
$= 1320 \times 10^3$ N

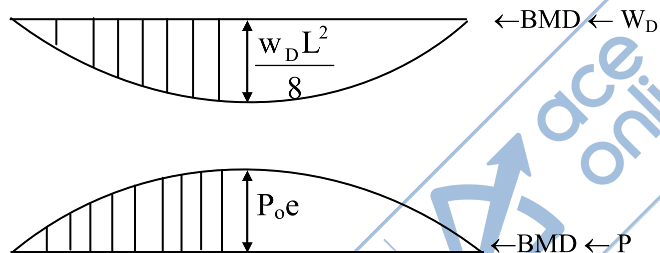
$= 1320$ kN

Final prestressing force $P = \eta.P = (1–0.18) \times 1320 = 1082.4$ kN

M.I of the section about c.g $I = \dfrac{bD^3}{12} = \dfrac{300\times600^3}{12} = 5.4\times10^9\,\text{mm}^4$

(i)





Deflection due to self weight of the beam

$$\Delta_1 = \frac{A\bar{x}}{EI} = \frac{\left(\frac{2}{3}.\frac{L}{2}.\frac{wDL^2}{8}\right)\left(\frac{5}{8}\frac{L}{2}\right)}{EI} = \frac{5}{384}\frac{w_D L^4}{E_C I}$$

$$= \frac{5}{384} \times \frac{4.5 \times (12000)^4}{35000 \times 5.4 \times 10^9}$$

$= 6.43$ mm (down ward)

Deflection due to prestressing cable

$$\Delta_2 = \frac{A\bar{x}}{EI} = \frac{\left(\frac{2}{3}.\frac{L}{2}.P_o e\right)\left(\frac{5}{8}\frac{L}{2}\right)}{E_C I} = \frac{5}{48}\frac{P_o e L^2}{E_C I}$$

$$= \frac{5}{48} \times \frac{1320 \times 10^3 \times 150 \times 12000^2}{35 \times 10^3 \times 5.4 \times 10^9}$$

$= 15.71$ mm (upward)

The net deflection at mid span due to self weight and initial prestress

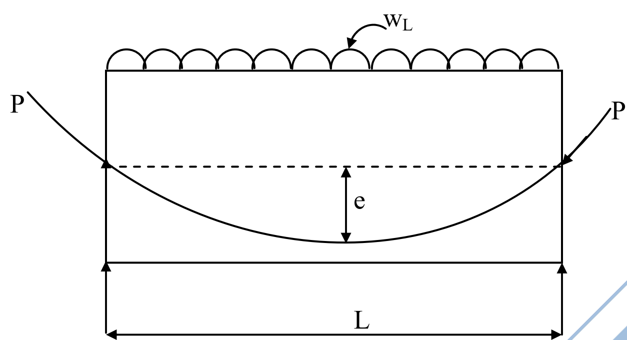
$\Delta = \Delta_1 - \Delta_2$

$= 6.43 - 15.71$

$= -9.28$ mm

$= 9.28$ mm (up ward)

(ii)



Deflection due to live load

$$\Delta_1 = \frac{5}{384}\frac{w_L L^4}{E_c I} = \frac{5}{384} \times \frac{20 \times 12000^4}{35 \times 10^3 \times 5.4 \times 10^9}$$

$= 28.57$ mm (down ward)

Deflection due to final rpestressing force

$$\Delta_2 = \frac{5}{48}\frac{PeL^2}{E_c I} = \frac{5}{48} \times \frac{1082.4 \times 10^3 \times 150 \times 12000^2}{35 \times 10^3 \times 5.4 \times 10^9}$$

$= 12.88$ mm (up ward)

Deflection due to self weight of beam

$\Delta_3 = 6.43$ mm (down ward)

Final deflection

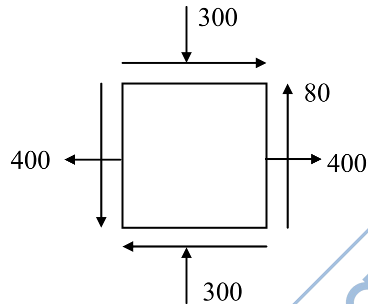
$\Delta = \Delta_1 - \Delta_2 + \Delta_3 = 28.57 - 12.88 + 6.43$

$= 22.12$ mm (down ward)

(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.

[10M]

Sol:



Principal stresses

$$\left.\begin{matrix}\sigma_1\\ \sigma_2\end{matrix}\right\} = \frac{\sigma_x+\sigma_y}{2} \pm \sqrt{\left(\frac{\sigma_x-\sigma_y}{2}\right)^2+\tau_{xy}^2}$$

$$= \frac{400-300}{2} \pm \sqrt{\left(\frac{400+300}{2}\right)^2+80^2} = 50 \pm 359$$

$\sigma_1 = +\,409$ MPa (ten)

$\sigma_2 = 50 - 359 = (-)\ 309$ MPa (comp)

Principal planes

$$\tan(2\theta_P) = \frac{2\tau_{xy}}{\sigma_x-\sigma_y} = \frac{2(80)}{400-(-300)}$$

$\theta_P = 6.43^\circ$

Angle of major principal plane, $\theta_P = 6.43^\circ$

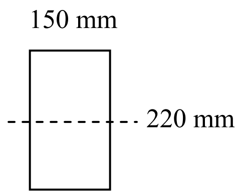
Angle of minor principal plane, $\theta_P + 90 = 6.43 + 90 = 96.43^\circ$ (or)

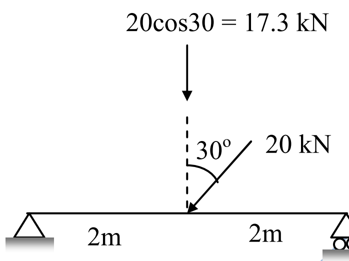
Maximum shear stress, $\tau_{max} = \dfrac{\sigma_1-\sigma_2}{2} = 359$ MPa

(c).(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.

[10M]

Sol:





$$\text{max BM, } M = \frac{w\ell}{4} = \frac{17.3(4)}{4} = 17.3\,\text{kN.m}$$

$$z = \frac{150 \times 220^2}{6} = 1.21 \times 10^6$$

$$\left.\begin{array}{r}\sigma\,\text{tension}\\ \sigma\,\text{comp}\end{array}\right\} = \frac{M}{Z} = \frac{17.3 \times 10^6}{1.21 \times 10^6}$$

$$= 14.3\text{ MPa}$$

$\sigma_{tension}$ occurs at the bottom fibre and σ_{comp} occurs at the top fibre of mid span of the beam.

07. (a)

(i) What is ferrocement and fiber-reinforced concrete? Write their advantages and disadvantages **[8M]**

Sol: Ferrocement:

It is a specialized form of reinforced concrete composed of a rich cement mortar matrix reinforced with multiple layers of small-diameter wire meshes. It is typically used in thin sections and is known for its high tensile strength and versatility in shaping the structure.

The advantages of ferrocement include its excellent tensile strength, resistance to impact and cracking, lightweight nature, and adaptability to complex forms. However, it is labor-intensive, demands skilled workmanship, and may suffer from corrosion if not adequately protected.

Fiber-reinforced concrete:

This concrete is prepared by mixing discrete fibers of steel, glass or polymers during concrete preparation to enhance its structural integrity and resistance to cracking.

Fiber-reinforced concrete offers improved toughness, durability, and resistance to environmental stressors like freeze-thaw cycles. It also reduces permeability and enhances impact resistance. Nonetheless, it can be costlier due to fiber additives, may exhibit inconsistent properties if fibers are unevenly distributed, and require precise mix design and handling.

(a) (ii) Describe, in short, the various methods of proportioning concrete. [6M]

Sol: Concrete proportioning is a critical process that ensures the desired strength, durability, and workability of the mix. The primary methods include volume batching, where ingredients are measured using containers; weight batching, which involves weighing materials for accuracy; the absolute volume method, which calculates the volume of each component based on specific gravity and total desired volume; and the trial mix method, which involves preparing sample batches and refining the proportions based on performance outcomes. Each method has its own merits, with weight batching and absolute volume methods offering higher precision, especially in structural applications.

(a) (iii) Write, in short, the various advantages of RC structures over other masonry structures. [6M]

Sol: Reinforced concrete (RC) structures offer several advantages over traditional masonry constructions. Their superior strength and load-bearing capacity stem from the embedded steel reinforcement, which allows them to withstand greater stresses and dynamic loads. RC structures exhibit enhanced resistance to environmental hazards such as earthquakes, wind, and fire, making them more suitable for modern infrastructure. They also provide greater design flexibility, enabling architects and engineers to create complex and aesthetically appealing forms. Furthermore, RC structures tend to have a longer service life, reduced maintenance needs, and improved durability against cracking and deterioration, positioning them as a preferred choice in contemporary construction.

(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. [20M]

Sol: Given

l = 3m, b_s = 230 mm, LL = 3 kN/m^2, M30, Fe – 500

C. C= 20 mm

Assume thickness of slab

$$D = \frac{\text{span}}{20} = \frac{3000}{20} = 150\,\text{mm}$$

$$d = D - C.C - \frac{\phi}{2} \text{ (Assume 10 mm dia main steel)}$$

$$= 150 - 20 - \frac{10}{2} = 125 \text{ mm}$$

Effective span (L)

(i) $l + b_s$ = 3 + 0.23 = 3.23m

(ii) l + d = 3 + 0.125 = 3.125 m

∴ L = 3.125 m

Loads:

Self weight of slab = γD = 25 × 0.15 = 3.75 kN/m^2

Live load = 3 kN/m^2

Floor finishes = 1 kN/m^2

Total working load w = 3.75 + 3 + 1 = 7.75 kN/m^2

Factored load $w_u = 1.5w = 1.5 \times 7.75 = 11.625$ kN/m^2

Design BM

Factored BM per m width

$$m_u = \frac{w_u L^2}{8} = \frac{11.625 \times 3.125^2}{8} = 14.19 \text{ kNm}$$

Effective depth of slab required to resist BM

$$d = \sqrt{\frac{M_u}{R_u b}} = \sqrt{\frac{M_u}{0.133 f_{ck} b}}$$

$$= \sqrt{\frac{14.19 \times 10^6}{0.133 \times 30 \times 1000}}$$

$= 59.6$ mm < 125 mm $\therefore$ O.K

Adopt D = 150 mm and d = 125 mm > 59.6 mm

$\therefore$ it will be designed as URS

$\therefore$ Area of main steel required

$$A_{st} = \frac{0.5 f_{ck}}{f_y}\left[1 - \sqrt{1 - \frac{4.6 M_u}{f_{ck} b d^2}}\right] bd$$

$$= \frac{0.5 \times 30}{500}\left[1 - \sqrt{1 - \frac{4.6 \times 14.19 \times 10^6}{30 \times 1000 \times 125^2}}\right] 1000 \times 125$$

$= 270.9$ mm^2

Area of minimum steel

0.12% of bD

$\frac{0.12}{100} \times 1000 \times 100 = 120\text{mm}^2 < 270.9 \text{ mm}^2$ $\therefore$ O.K

Hearty Congratulations

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Area of max steel

0.04 bD

$0.04 \times 1000 \times 100 = 4000\ \text{mm}^2 > 270.9\ \text{mm}^2 \quad \therefore$ O.K

Spacing required for main steel

$$S = 1000\frac{a_{st}}{A_{st}}$$

$$= 1000\frac{a_{st}}{A_{st}}$$

$$= 1000 \times \frac{\frac{\pi}{4}\times 10^2}{270.9} = 289\ \text{mm}$$

Maximum spacing for main steel

$$\left.\begin{array}{l}\text{(i) } 3d = 3 \times 125 = 375\ \text{mm} \\ \text{(ii) } 300\ \text{mm}\end{array}\right\}\ \text{Smaller}$$

$S < 300\ \text{mm} \quad \therefore$ O . K

Provide 10 mm ϕ @ 280 mm c/c

Distribution steel

Provide min steel = 120 mm^2

Spacing for distribution steel

$$S = 1000\frac{a_{st}}{A_{st}} = 1000 \times \frac{\frac{\pi}{4}\times 8^2}{120} = 418\ \text{mm}$$

Maximum spacing for distribution steel

$$\left.\begin{array}{l}\text{(i) } 5d = 5\times 75 = 375\text{ mm}\\ \\ \text{(ii) } 300\text{ mm}\end{array}\right\}\text{ Smaller}$$

S > 300 mm

Provide 8 mm ϕ @ 300 mm c/c

Check for deflection

$$\left(\frac{L}{d}\right)_a = \frac{3100}{125} = 24.8$$

$$\left(\frac{L}{d}\right)_p = \alpha\beta\gamma\delta\lambda = 20\times 1\times 1.3\times 1\times 1 = 26$$

$\alpha = 20 \rightarrow$ for simply supported

$\beta = 1 \rightarrow$ span < 10 m

$\delta = 1 \rightarrow$ singly reinforced

$\lambda = 1 \rightarrow$ rectangular c/c

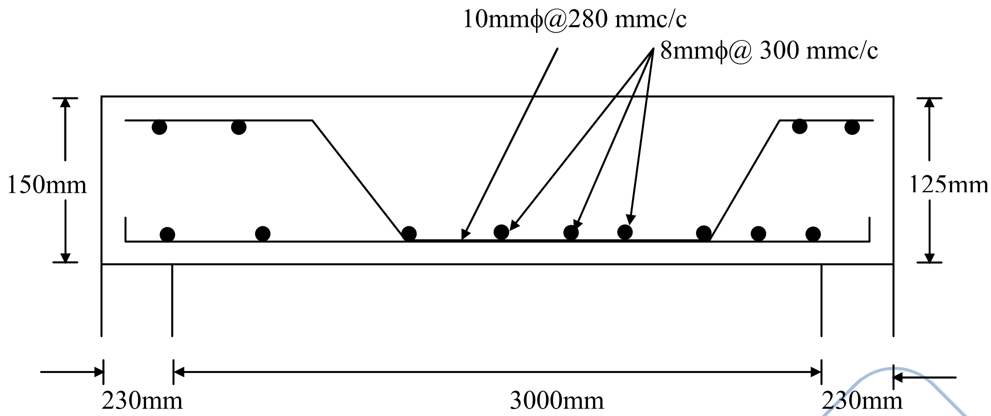
$\gamma = 1.3 \rightarrow$ from graph

$$p_t = \frac{100A_{st}}{bd} = \frac{100\times\left(\dfrac{1000\times\frac{\pi}{4}\times 10^2}{280}\right)}{1000\times 125} = 0.22$$

$$f_s = 0.58f_y\frac{A_{st_R}}{A_{st_P}}$$

$$= 0.58\times 500\times\frac{270.9}{280.5} = 280\text{ N/mm}^2$$

$$\left(\frac{L}{d}\right)_a < \left(\frac{L}{d}\right)_P \qquad \therefore \text{ Safe in deflection}$$


10mmϕ@280 mmc/c
8mmϕ@ 300 mmc/c
150mm
125mm
230mm
3000mm
230mm

IS 456 : 2000

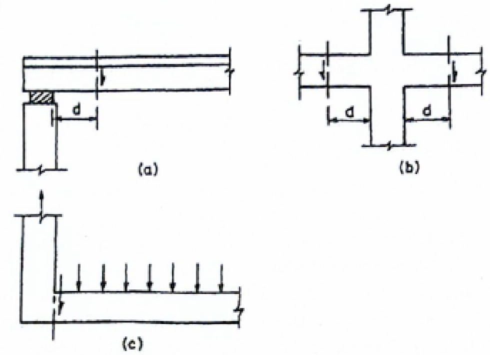


FIG. 2 TYPICAL SUPPORT CONDITIONS FOR LOCATING FACTORED SHEAR FORCE

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:

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

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

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.

IS 456 : 2000

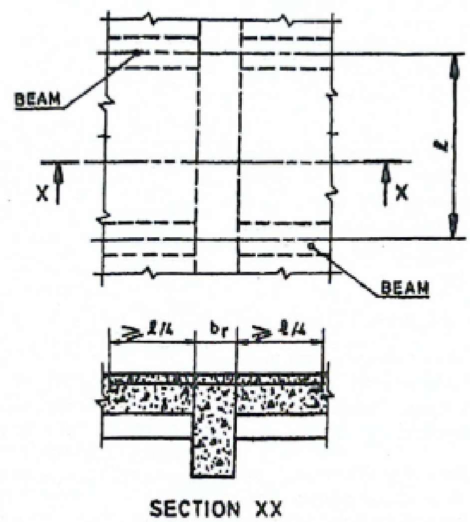
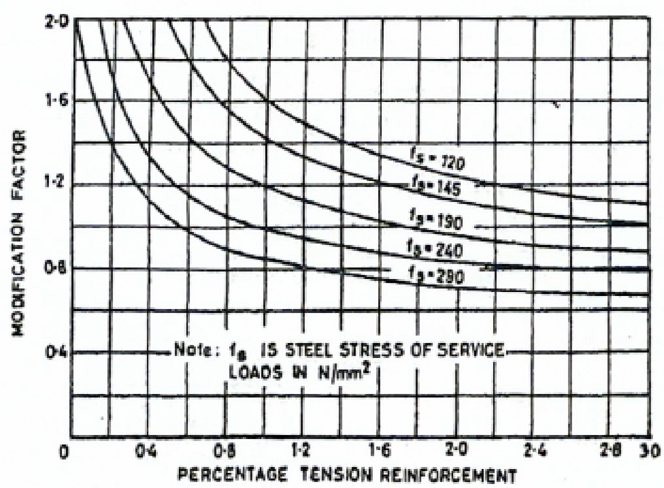


FIG. 3 TRANSVERSE REINFORCEMENT IN FLANGE OF T-BEAM WHEN MAIN REINFORCEMENT OF SLAB IS PARALLEL TO THE BEAM

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.



$$f_s = 0.58\, f_y \frac{\text{Area of cross-section of steel required}}{\text{Area of cross-section of steel provided}}$$

FIG. 4 MODIFICATION FACTOR FOR TENSION REINFORCEMENT

IS 456 : 2000

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/mm²	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_{st} limited to 165 N/mm² in working stress design.

IS 456 : 2000

ANNEX G

(Clause 38.1)

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 notutral 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 :

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

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 :

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

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

$$0.003\,5\, \frac{(x_{u,max} - d')}{x_{u,max}}$$

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 SE-CTION

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 GN/m^2 and $\frac{1}{m} = 0.3$ **[20M]**

Sol: $\sigma_1 = 60$ MPa;

$\sigma_2 = 48$ MPa

$\sigma_3 = -36$ MPa

(i) total strain energy

$$U = \frac{1}{2E}\left(\sigma_1^2 + \sigma_2^2 + \sigma_3^2 - 2\mu\left(\sigma_1\sigma_2 + \sigma_2\sigma_3 + \sigma_3\sigma_1\right)\right)$$

$$= \frac{1}{2\times 200\times 10^3}\left[60^2 + 48^2 + 36^2 - 2\times 0.3(60\times 48 - 48\times 36 - 36\times 60)\right]$$

U = 0.0195 N.mm

(ii) Volumetric strain energy

Assuming volume, V = 1 mm^3

$\therefore$ Volumetric strain energy = 0.0195 N-mm/mm^3

(iii) Shear strain energy (or) Distortional strain energy

$$U_{shear} = \frac{1}{12G}\left[(\sigma_1 - \sigma_2)^2 + \left(\sigma_2 - \sigma_3^2\right) + (\sigma_3 - \sigma_1)^2\right]$$

$$G = \frac{E}{2(1+\mu)} = \frac{200}{2(1+0.3)} = 77\,\text{GPa}$$

$$U_{shear} = \frac{1}{12\left(77\times 10^3\right)}\left[(60-48)^2 + (48+36)^2 + (60+36)^2\right] = 0.178$$

(iv) Factor of safety on total strain energy criteria.

$$\sigma_1^2+\sigma_2^2+\sigma_3^2-2\mu(\sigma_1\sigma_2+\sigma_2\sigma_3+\sigma_3\sigma_1)=\left(\frac{f_y}{F_S}\right)^2$$

$$0.0195(2\times 200\times 10^3)=\left(\frac{120}{F_S}\right)^2$$

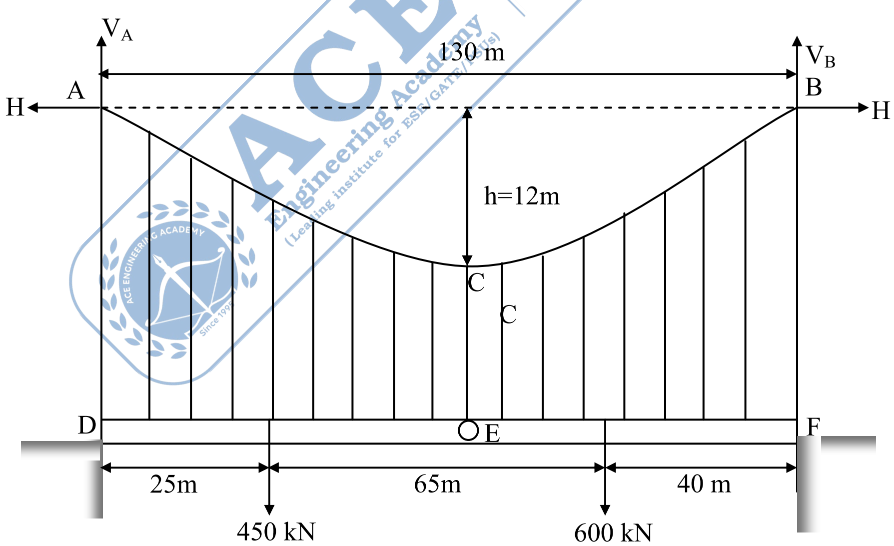
$\therefore F_S = 1.36$

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

(i) Maximum tension in the cable

(ii) Shear force and bending moment values for girder at 40 m from the left support [10M]

Sol:



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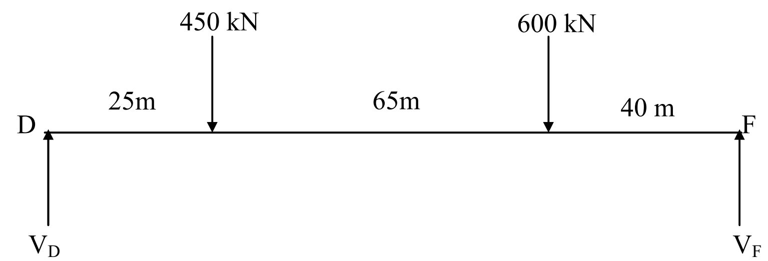
Reaction:



$V_D + V_F = 450 + 600 \quad \rightarrow(1)$

Taking moment about 'D'

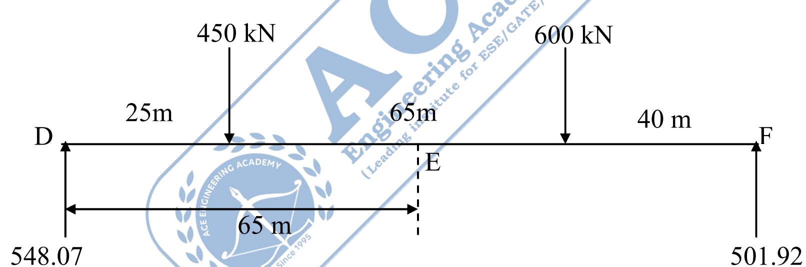
$\Sigma M_D = 0$ -ve +ve

$-V_F \times 130 + 600 \times 90 + 450 \times 25 = 0$

$V_F = 501.92$ kN

$V_D = 548.07$ kN

Horizontal reaction at each end of the cable



$$H = \frac{M_{SE}}{h}$$

Beam moment at 'E' the middle point of the girder

Sagging +ve

Hogging –ve

$M_{SE} = + 548.07 \times 65 - 450 \times 40$

$= 17, 624.55$ kN-m

$$H = \frac{17,624.55}{12} = 1468.71\,kN$$

Udl transferred to the cable be 'w_e' per unit run

$$H = \frac{w_e L^2}{8h}$$

$$1468.71 = \frac{w_e \times 130^2}{8 \times 12}$$

$w_e = 8.34$ kN/m

Vertical reaction at each supp9ort for the cable

$$V = \frac{w_e L}{2} = \frac{8.34 \times 130}{2} = 542.1\,kN$$

Maximum tension in the cable

$$T_{max} = \sqrt{H^2 + V^2} = \sqrt{(1468.71)^2 + (542.1)^2} = 1565.56\ kN$$

Shear force at any section of the girder.

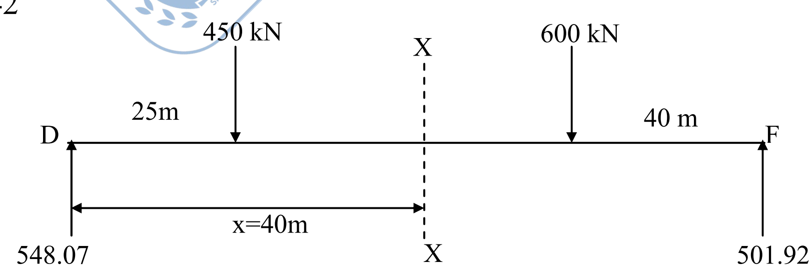
$S = S_x - H \tan\theta$

$$\tan\theta = \frac{4h}{L^2}(L - 2x)$$

$\therefore$ At 40 m from left end

$$\tan\theta = \frac{4 \times 12}{130^2}(130 - 2 \times 40)$$

$\tan\theta = 0.142$



$S_x = 548.07 - 450 = 98.07$ kN

Actual SF at 40 m from left end

$S = 98.07 - 1468.71 \times 0.142 = -110.48$ kN

Actual BM at any section of the girder

$M = M_x - H_y$

$$y_{40m} = \frac{4hx}{L^2}(L - x) = \frac{4 \times 12 \times 40}{130^2}(130 - 40) = 10.22 \text{ m}$$

Beam moment at 40 m from left end

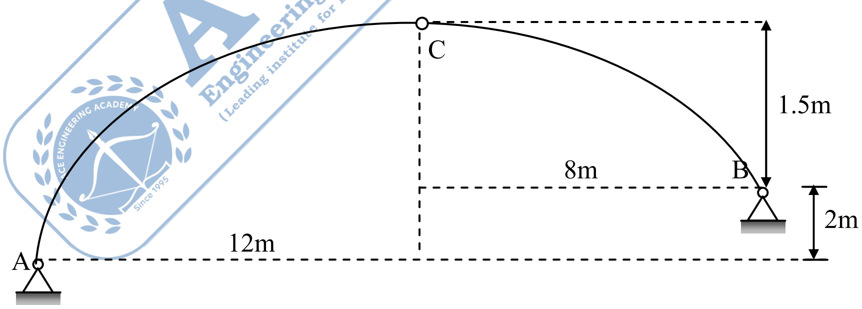
$M_x = 548.07 \times 40 - 450 \times 20 = 12922.8$ kN-m

$M = 12922.8 - 1468.71 \times 10.22 = -2087.41$ kN-m

08. (a).(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.

[10M]

Sol:



Note: Since no external loads or moments are applied and the member is free of support reactions, the bending moment is zero throughout the structure.

(b). A rectangular cantilever reinforced concrete (RC) beam of 300 mm × 600 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, $E_c = 500\sqrt{f_{ck}}$. Flexural strength of concrete $f_{cr} = 0.7\sqrt{f_{ck}}$.

Relevant portion of the IS 456 is enclosed.

[20M]

Sol: Given:

b =300 mm, D = 600 mm, L =3m, DL = 35 kN/m, LL = 1.5 kN/m

A_{st} : 4–25 mmϕ, A_{sc} : 2–20 mmϕ, $e_c = d' = 50$ mm, M25, Fe – 500, ϕ : 1.6, $E_c = 5000\sqrt{f_{ck}}$,

$f_{cr} = 0.7\sqrt{f_{ck}}$

Effective depth of beam

$d = D - e_c$

$= 600 - 50$

$= 550$ mm

Effective modulus of elasticity

$$E_{ce} = \frac{E_C}{1+\theta} = \frac{5000\sqrt{f_{ck}}}{1+\theta} = \frac{5000\sqrt{25}}{1+1.6} = 9615.4 \text{ N/mm}^2$$

Modular ratio

$$m = \frac{E_s}{E_{ce}} = \frac{2\times10^5}{9615.4} = 20.8$$

Actual depth of N.A

$$\frac{bx^2}{2}+(1.5m-1)A_{sc}(x-d')=mA_{st}(d-x)$$

$$\frac{300x^2}{2}+(1.5\times20.8-1)\times2\times\frac{\pi}{4}\times20^2(x-50)=20.8\times4\times\frac{\pi}{4}\times25^2(550-x)$$

$$150x^2+18.975\times10^3x-948.76\times10^3=22.46\times10^6-40.84\times10^3x$$

$$150x^2+59.815\times10^3x-23.408\times10^6=0$$

$x = 243.11$ mm

Section modulus

$$Z=d-\frac{x}{3}=550-\frac{243.11}{3}=468.96\,mm$$

Moment of inertia of cracked section

$$I_r=\frac{bx^3}{3}+1.5mA_{sc}(x-d')^2+mA_{st}(d-x)^2$$

$$=\frac{300\times243.11^3}{3}+1.5\times20.8\times2\times\frac{\pi}{4}\times20^2(243.11-50)^2+20.8\times4\times\frac{\pi}{4}\times25^2(550-243.11)^2$$

$= 1.437 \times 10^9 + 731.04 \times 10^6 + 3.846 \times 10^9$

$= 6.01 \times 10^9\ mm^4$

M.I of gross c/s

$$I_{gr}=\frac{bD^3}{12}=\frac{300\times600^3}{12}=5.4\times10^9\ mm^4$$

Cracking moment

$$M_{cr}=\frac{f_{cr}}{y_t}I_{gr}$$

$$=\frac{0.7\sqrt{25}}{\frac{600}{2}}\times5.4\times10^9$$

$= 63 \times 10^6$ Nmm $= 63$ kNm

Maximum Bending Moment due to dead load

$$M = \frac{w_D L^2}{2} = \frac{35 \times 3^2}{2} = 157.5\,\text{kNm}$$

Effective moment of inertia

$$I_{eff} = \frac{I_r}{1.2 - \frac{M_{cr}}{M}.\frac{Z}{d}\left(1 - \frac{x}{d}\right)\frac{b_w}{b}}$$

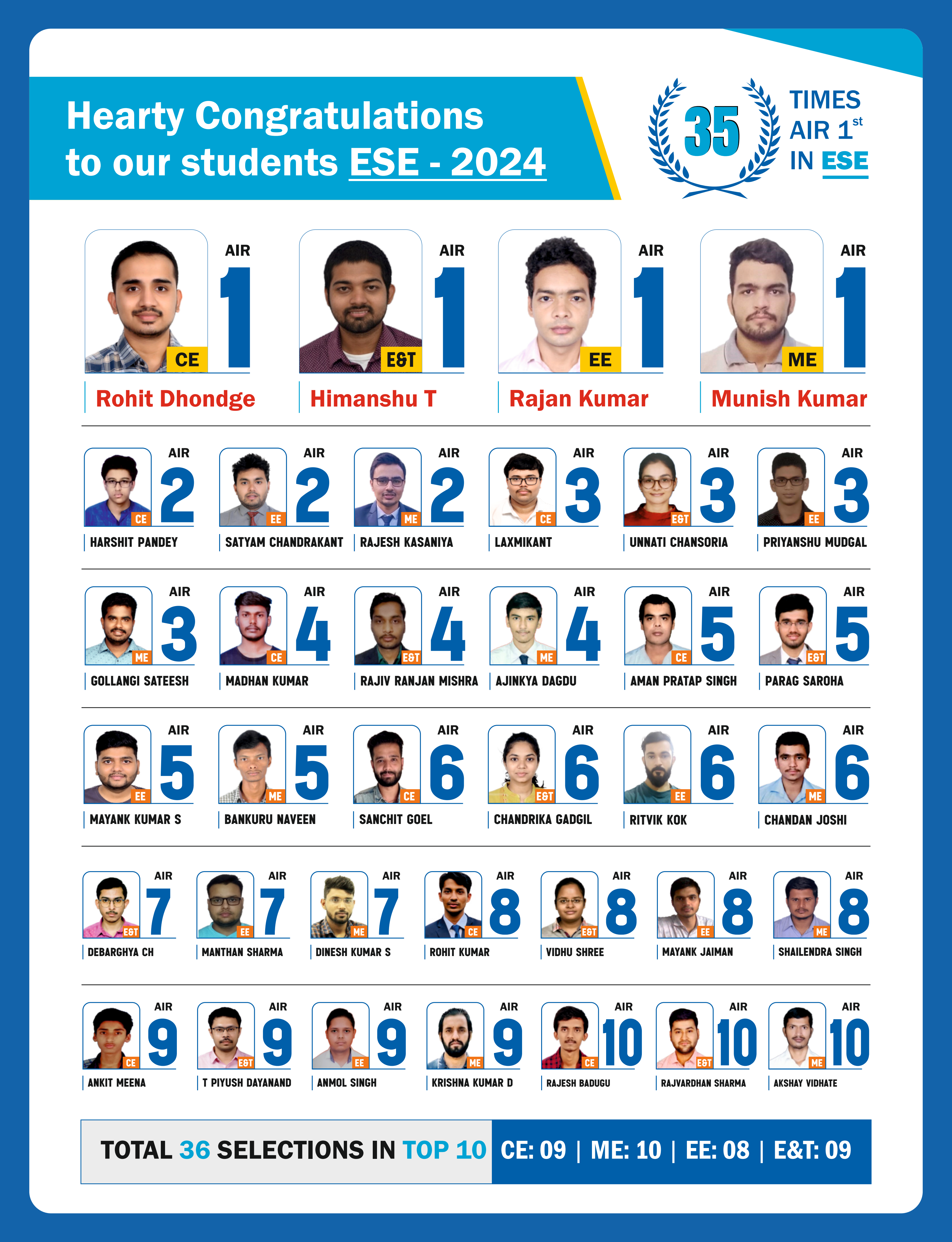
$$= \frac{6.01 \times 10^9}{1.2 - \frac{63 \times 10^6}{157.5 \times 10^6} \times \frac{468.96}{550}\left(1 - \frac{243.11}{550}\right) \times 1}$$

$= 5.95\times 10^6\ \text{mm}^4 > I_{gr}$

Hence $I_{eff} = I_{gr} = 5.4\times 10^9\ \text{mm}^4$

Initial plus creep deflection due to permanent loads

$$\left(\Delta_{i+cc}\right)_{perm} = \frac{w_D L^4}{8E_{ce} I_{eff}} = \frac{35 \times 3000^4}{8 \times 9615.4 \times 5.4 \times 10^9} = 6.82\ \text{mm}$$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AIR 7 EE MANTHAN SHARMA
AIR 7 ME DINESH KUMAR S
AIR 8 CE ROHIT KUMAR
AIR 8 E&T VIDHU SHREE
AIR 8 EE MAYANK JAIMAN
AIR 8 ME SHAILENDRA SINGH
AIR 9 CE ANKIT MEENA
AIR 9 E&T T PIYUSH DAYANAND
AIR 9 EE ANMOL SINGH
AIR 9 ME KRISHNA KUMAR D
AIR 10 CE RAJESH BADUGU
AIR 10 E&T RAJVARDHAN SHARMA
AIR 10 ME AKSHAY VIDHATE
TOTAL 36 SELECTIONS IN TOP 10
CE: 09 | ME: 10 | EE: 08 | E&T: 09

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:

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

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

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:

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

where

X_e = modified value of X,

X_1, X_2 = values of X at the supports,

X_o = 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 $k_4 \frac{\varepsilon_{cs}}{D}$

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

$$k_4 = 0.72 \times \frac{p_t - p_c}{\sqrt{p_t}} \le 1.0 \text{ for } 0.25 \le p_t - p_c < 1.0$$

$$= 0.65 \times \frac{p_t - p_c}{\sqrt{p_t}} \le 1.0 \text{ for } p_t - p_c \ge 1.0$$

Table 25 Values of Coefficient, k_1

(*Clause* C-2.1)

k_2	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_2 is given by

$$k_2 = \frac{M_1 + M_2}{M_{F1} + M_{F2}}$$

where

M_1, M_2 = support moments, and

M_{F1}, M_{F2} = fixed end moments.

IS 456 : 2000

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:

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

where

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

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

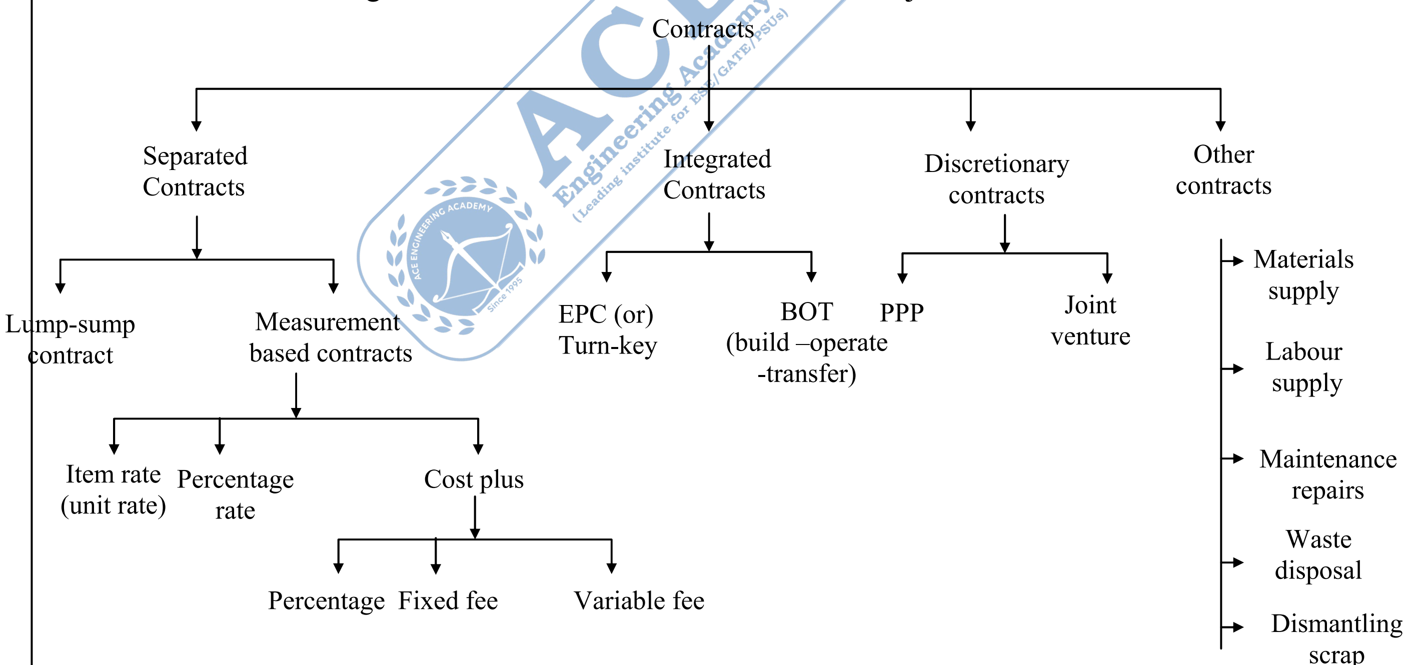
and

$a_{i(perm)}$ = 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? [10M]

Sol: Contract:

- It is an agreement between two parties which is enforceable by law
- Contract is a legal document and it is known of lawful object.



- Tendering is a process in which the project owner/client invite bids from potential suppliers/contractors for the purpose of engineering, procurement and construction works.
- The Tendering process has following stages

  1. Preparation of tender documents
  2. Invitation to Tender (Notice Inviting Tender)
  3. Bid submission
  4. Bid evaluation
  5. Contract award

- The following documents/details need to be provided in tender documents

  i. Name of the authority inviting bids/Tenders

  ii. Name of the project and its details

  iii. Conditions for eligibility of contracting agencies to submit bids

  iv. Estimated cost and time of completion of the project

  v. Technical specification and drawings

  vi. General conditions of contraction (GCC) and special conditions of contract (SCC)

(c).(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 **[10M]**

Sol:

Effective Loader Capacity	=	Loader Capacity × Bucket Fill Factor × Swell Factor
	=	$2.3\ m^3 \times 0.85 \times 0.90 = 1.759\ m^3$
Total Available Working Hours	=	Project Duration × Yearly Working Hours
	=	5 years × 2,000 hr/year
	=	10,000 hours
Effective Working Hours	=	Total Hours × Operating Efficiency × Job Management
Factor	=	$10{,}000 \times 0.85 \times 0.70 = 5{,}950$ hours
Required Material per Hour	=	Total Quantity ÷ Effective Working Hours
	=	$5{,}000{,}000\ m^3 \div 5{,}950\ hr = 840.34\ m^3/hr$
Loader Cycle Time	=	0.50 minutes = 0.008 hours
Cycles per Hour	=	$1 \div 0.008$ = 120 cycles/hr
Loader Productivity	=	$120\ cycles/hr \times 1.759\ m^3$
	=	$211.14\ m^3/hr$
Required Number of Loaders	=	$840.34 \div 211.14 = 3.98$
	=	**4 loaders**

The dumper cycle time includes:

• Loaded haul time	=	6 km ÷ 20 km/hr	=	0.300 hr
• Empty return time	=	6 km ÷ 25 km/hr	=	0.240 hr
Total Dumper Cycle Time	=	0.300 + 0.240	=	0.54 hours
Effective Dumper Capacity	=	$30\ m^3 \times 0.90$	=	$27.0\ m^3$
Cycles per Hour	=	$1 \div 0.54$	=	1.85 cycles/hr
Dumper Productivity	=	$1.85\ cycles/hr \times 27.0\ m^3$	=	$49.95\ m^3/hr$
Required Number of Carriers	=	$840.34 \div 49.95$		

= 16.82

= 17 carriers.