# RPSC AEn-2024 Main Test Series

## CIVIL ENGINEERING

## Test 8

Test Mode : • Offline • Online

## Subjects : Steel Structure + RCC

## DETAILED EXPLANATIONS

1. **Solution:**

**Neutral Axis:** It is an imaginary axis formed by intersection of neutral plane with cross-section of beam. Neutral axis divides the cross-section of a beam into the tension and compression zones lying on the opposite side of the plane.

2. **Solution:**

Bond strength ($\tau_{bd}$) is the shear stress developed along the contact surface between the reinforcing steel and the surrounding concrete, which prevents the bar from slipping out of concrete.

3. **Solution:**

For reducing the length of the gusset plates, an additional short angle may be used to reduce the joint length which in turn reduces the shear lag also such an angle is called the lug angle.

4. **Solution:**

Given: Height of pole, $L = 5$ m

Effective length of pole, $l_{eff} = kL = 1.2 \times 5 = 6$ m

**5. Solution:**

Following are the types of failure of tension member:
- Gross section yielding.
- Due to net section rupture.
- Block shear failure.

**6. Solution:**

A column will be considered as long when the ratio of the effective length to its lateral dimension is greater than or equal to 12.

**7. Solution:**

The modular ratio ($m$) is the ratio of the modulus of elasticity of steel ($E_s$) to that of concrete ($E_c$), used to convert the area of steel into an equivalent area of concrete in transformed-section analysis.

$$m = \frac{E_s}{E_c}$$

It represents how much stiffer steel is compared to concrete and helps in sharing of stresses between the two materials.

**8. Solution:**

High strength concrete is necessary is prestressed concrete as the material offers high resistance in tension, shear, bond and bearing. High strength concrete is less liable to shrinkage cracks and has a higher modulus of elasticity and smaller ultimate creep strain, resulting is a smaller loss of prestress.

**9. Solution:**

Prestressing force,

$$P_2 = P_o e^{-(\mu\alpha + kx)}$$

$P_o$ = Prestressing force at the jacking end.

$\mu$ = Coefficient of friction between cable and duct.

$\alpha$ = Cumulative angle in radians between any two tangents drawn from the cable profile under consideration.

$K$ = Friction coefficient for wave effect

**10. Solution:**

Bracing is required to resist horizontal loading in pin-jointed buildings, including roof trusses. Bracing of roof trusses and supporting columns provide still rigid structure. When wind blows normal to the inclined surface of the trusses, it is efficiently resisted by all the members of the truss and the wind forces are transferred to the supports at the ends of the truss.

**11. Solution:**

**Assumptions:** Following are the assumptions for design of reinforcement concrete section by limit state method:

1. Plane sections normal to the axis remain plane after bending.
2. The maximum strain in concrete at the outermost compression fibre is taken as 0.0035 in bending.
3. The relationship between the stress-strain distribution in concrete is assumed to be parabolic. For design purpose, the compressive strength of concrete is assumed to be 0.67 times the characteristic strength of concrete. The partial safety factor ($\gamma_{mt}$) = 1.5 shall be applied in addition to this.
4. Maximum compressive stress in concrete

$$= \frac{0.67 f_{ck}}{1.5} = 0.45 f_{ck}$$

where, $f_{ck}$ = Characteristic strength of concrete.

5. The tensile strength of the concrete is ignored.
6. The stresses in the reinforcement are taken from the stress-strain curve for the type of steel used. For design purpose, the partial safety factor ($\gamma_{ms}$) equal to 1.15 shall be applied.
7. The maximum strain in the tension reinforcement in the section at failure shall not be less than

$$\in_{st} \geq \frac{f_{ck}}{1.15 E_s} + 0.002$$

$f_y$ = Characteristics strength of steel

$E_s$ = Modulus of elasticity of steel

**12. Solution:**

**Given:**

Width of beam, $b = 300$ mm

Effective depth of beam, $d = 500$ mm

Diameter of bars = 18 mm

Number of bars = 4

$f_{ck} = 20 \text{ N/mm}^2$

$f_y = 250 \text{ N/mm}^2$

Depth of neutral axis. $x_u = ?$

Area of steel, $A_{st} = 4 \times \frac{\pi}{4} \times 18^2 = 1017.88 \approx 1018 \text{ mm}^2$

Let steel yields i.e. $f_{st} = 0.87 f_y$

Equating tensile and compressive forces,

$$C = T$$

$$\Rightarrow \quad 0.36 f_{ck}\, b\, x_u = 0.87 f_y A_{st}$$

$$\Rightarrow \quad 0.36 \times 20 \times 300x = 0.87 \times 250 \times 1018$$

$$\Rightarrow \quad x_u = 102.5 \text{ mm}$$

Limiting depth of neutral axis, $x_{u\,\text{lim}} = 0.53\, d$ (For Fe250)

$= 0.53\,(500) = 265 \text{ mm} < x_u\ ( = 102.5 \text{ mm})$

= Section is under-reinforced and $f_{st} = 0.87 f_y$ is correct.

**13. Solution:**

**One way slab**

1. $(l_y/l_x) > 2.0$
2. The bending takes place in one direction only i.e., along shorter span.
3. Depth required is more.
4. Main steel reinforcement is provided along shorter span.
5. Less economical as thickness is more and the amount of steel required is also more.

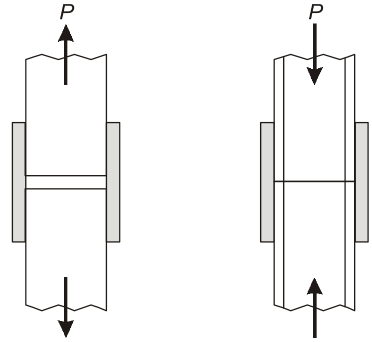
**Two way slab**

1. $(l_y/l_x) \leq 2.0$
2. The bending takes place in both directions.
3. Depth required is less.
4. Main steel reinforcement is provided along both the directions.
5. More economical as the thickness of slab is less and the amount of steel required is also less.

**14. Solution:**

**Splice design:** Splice is provided where we need to join two members. Whenever a splice occurs, the force is transferred from member to splice through connection. This force is again transferred to other connecting member through connection.

Depending on the nature of axial force, splice can either be tension splice or the compression splice.




Generally the exact load carried by a splice is uncertain. Thus tension splice is designed for a design load or 0.3 times the member design capacity, whichever is more. It is always beneficial to design the splice for the strength of the member.

**15. Solution:**

**Torsion in RCC beams:** Reinforced concrete sections are also subjected to torsional moments which cause twisting or warping of the section.

The reinforcements for the beam consist of the following:

(i) Longitudinal reinforcement determined for an equivalent ultimate bending moment which is based on the actual bending moment and equivalent moment due to torsion.

(ii) Web reinforcement determined for an equivalent ultimate shear which is based on the actual shear and equivalent shear due to torsion.

1. **Equivalent shear force:** The equivalent ultimate shear is given by

$$V_e = V_u + 1.6\frac{T_u}{b}$$

where, $V_u$ = Ultimate shear force

2. **Equivalent moment:** The equivalent ultimate moment is given by

$$M_e = M_u + M_t$$

and $$M_t = T_u\left[\frac{1+\frac{D}{b}}{1.7}\right]$$

where, $M_u$ = Actual ultimate moment

$T_u$ = Ultimate torsion

$D$ = Overall depth of the beam.

$b$ = Width of the beam

**16. Solution:**

Given:

$$B = 300 \text{ mm}, \; d = 580 \text{ mm}, \; V_u = 440 \text{ kN}$$

Concrete used is M30 and steel is Fe415

1. $V_{uc} = \tau_c Bd = 0.66 \times 300 \times \dfrac{580}{1000} \text{kN} = 114.84 \text{ kN}$

2. $V_{us} = V_u - V_{uc} = 440 - 114.84 \text{ kN} = 325.16 \text{ kN}$

3. Spacing of 2-legged 10 mm diameter stirrups,

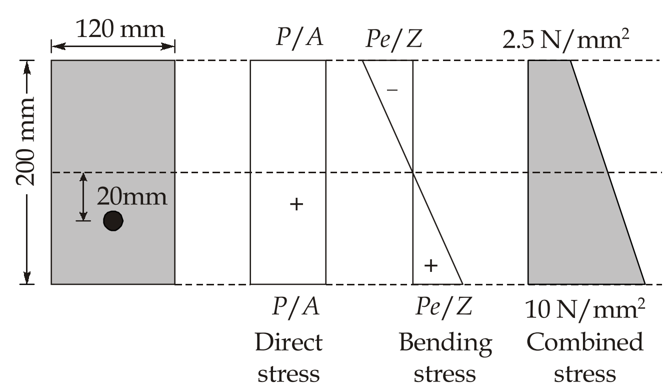
$$S_v = \frac{A_{sv} \times 0.87 f_y \times d}{V_{su}}$$

$$\Rightarrow \quad S_v = \frac{2 \times \frac{\pi}{4} \times (10)^2 \times 0.87 \times 415 \times 580}{325.16 \times 1000}$$

$$\Rightarrow \quad S_v = 101.16 \text{ mm} \simeq 100 \text{ mm (say)}$$

$\therefore$ Provide 2 legged 10 mm diameter stirrups @ 100 mm c/c.

**17. Solution:**




$$\sigma_{top} = \frac{P}{A} - \frac{Pe}{Z}$$

$$\Rightarrow \quad \sigma_{top} = \frac{150 \times 10^3}{120 \times 200} - \frac{150 \times 10^3 \times 20 \times 6}{120 \times (200)^2}$$

$$\Rightarrow \quad \sigma_{top} = 6.25 - 3.75$$

$$\Rightarrow \quad \sigma_{top} = 2.5 \text{ N/mm}^2 \text{ (compression)}$$

$$\sigma_{bottom} = \frac{P}{A} + \frac{Pe}{Z} = 6.25 + 3.75$$

$$\Rightarrow \quad \sigma_{bottom} = 10 \text{ N/mm}^2 \text{ (compression)}$$

**18. Solution:**

**Limit State:** For a structure, limit state is a state of impending failure, beyond which the structure cannot perform its intended function satisfactorily.

Beyond limit state, the structure cannot be used either due to collapse of structure or its unserviceability.

**There are two types of limit state:**

1. **Ultimate limit state (limit state of collapse):**

   It is due to failure of structure in any of the following modes:

   (a) flexure (b) compression

   (c) torsion (d) shear

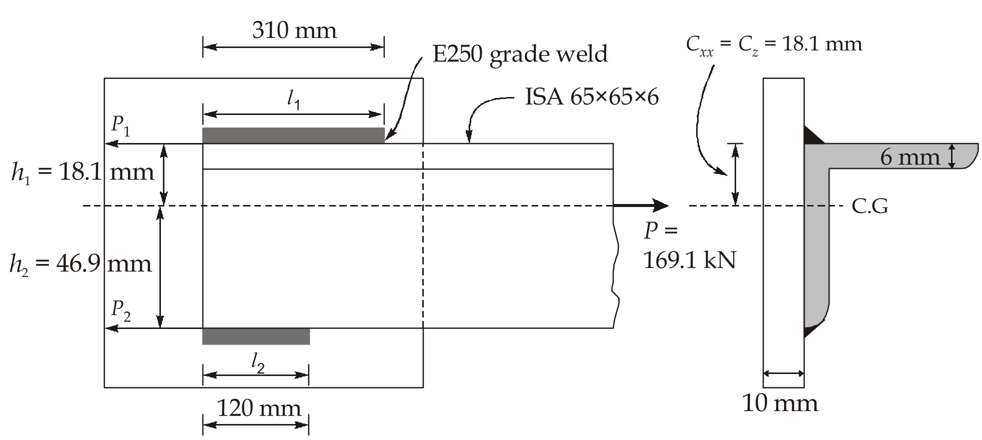
2. **Serviceability limit state:**

   In this structure becomes unserviceable due to attainment of the limit state in any of the following:

   (a) excessive deflection (b) excessive cracking

   (c) vibration (d) corrosion

**19. Solution:**




For ISA 65 × 65 × 6,

$$A = 744 \text{ mm}^2$$

$$C_z = 18.1 \text{ mm}$$

$$\text{Maximum size of weld} = \frac{3}{4} \times 6 = 4.5 \text{ mm}$$

Minimum size of weld for a plate of 10 mm thickness = 3 mm

So, provide. Size of weld, $\alpha = 3$ mm [Always provide minimum size of weld]

$P_T$ = Factored tensile strength of angle section

$$= A_g \times \frac{f_y}{1.1} = 744 \times \frac{250}{1.1} \text{N} = 169.1 \text{ kN}$$

$\therefore$ $P_T = 169.1$ kN

**Analysis :** Finding $P_1$ and $P_2$

$$\Sigma F_x = 0$$

$\Rightarrow$ $-P_1 - P_2 + P = 0$

$\Rightarrow$ $P_1 + P_2 = 169.1$ kN ...(i)

$$\Sigma M_{\text{C.G. line}} = 0$$

$\Rightarrow$ $-P_1 \times h_1 + P_2 \times h_2 = 0$

$\Rightarrow$ $-P_1 \times 18.1 = -P_2 \times 46.9$

$\Rightarrow$ $P_1 = 2.59\,P_2$ ...(ii)

From equation (i) and (ii)

$$2.59\,P_2 + P_2 = 169.1$$

$\Rightarrow$ $P_2 = 47.1$ kN

$\therefore$ $P_1 = 2.59 \times 47.1 = 121.989 \simeq 122$ kN

**Design :** Finding $l_1$ and $l_2$

$$P_1 = f_s \times l_1 \times t_t$$

$\Rightarrow$ $$122 \times 10^3 \text{ N} = \frac{410}{\sqrt{3} \times 1.25} \times l_1 \times 0.7 \times 3$$

$$f_s = \frac{f_u}{\sqrt{3} \times 1.25}$$ for E 250 grade, $f_u = 410$ MPa

Throat thickness, $t_t = 0.7s = 0.7 \times 3 = 2.1$ mm

$\therefore$ $l_1 = 306.78$ mm

$\therefore$ Provide effective length, $l_1 = 310$ mm

$$P_2 = f_s \times l_2 \times t_t$$

$\Rightarrow$ $$47.1 \times 10^3 \text{ N} = \frac{410}{\sqrt{3} \times 1.25} \times l_2 \times 0.7 \times 3$$

$\Rightarrow$ $l_2 = 118.4$ mm

$\therefore$ Provide effective length, $l_2 = 120$ mm

**Note :** Always welds must begin from the end of angle section as shown in figure.

**20. Solution:**

**Step-1:** Checking the column as short or long

Unsupported length of column

$$L_0 = 3.0 \text{ m} = 3000 \text{ mm}$$

Effective length of column, $(l_e)_x = (l_e)_y = 3000 \times 1.2 = 3600$ mm

$$D_x = D_y = 400 \text{ mm}$$

Slenderness ratio, $\lambda = \dfrac{l_e}{D} = \dfrac{3600}{400} = 9 < 12$

Slenderness ratio is less than 12 so the column is short.

**Step-2:** Calculation of minimum eccentricity

$$(e_x)_{min} = (e_y)_{min} = \max\begin{cases} \dfrac{l_x}{500} + \dfrac{D}{30} = \dfrac{3000}{500} + \dfrac{400}{30} = 19.33 \text{ mm} \\ 20 \text{ mm} \end{cases}$$

$$= 20 \text{ mm}$$

Also $0.05B = 0.05 \times 400 = 20$ mm

So, $e_{min} \not> 0.05B$

Since, the minimum eccentricities are less than 0.05 times the least lateral dimension of column the following formula given by IS 456 : 2000 can be used for the design of axially loaded short columns.

**Step-3:** Use of column design formula

$$P_u = 0.4 f_{ck} A_g + \left(0.67 f_y - 0.4 f_{ck}\right) A_{sc}$$

Factored axial load, $P_u = 1.5 \times 1500 = 2250$ kN

$\therefore$ $2250 \times 10^3 = (0.4 \times 25 \times 400^2) + (0.67 \times 500 - 0.4 \times 25)\ A_{sc}$

$\Rightarrow$ $A_{sc} = 2000 \text{ mm}^2$

**Step-4:** Reinforcement design

Provide (4-16 mm $\phi$) + (4 - 20 mm $\phi$) bars with 20 mm diameter bars at the corner and 16 mm diameter bars at the face.

$$(A_{sc})_{provided} = 2060.9 \text{ mm}^2 > 2000 \text{ mm}^2$$

Percentage of reinforcement provided = $2060.9 \times \dfrac{100}{400^2} = 1.288\%$

> Minimum reinforcement required (= 0.8%)

< Maximum reinforcement permissible (= 6%)

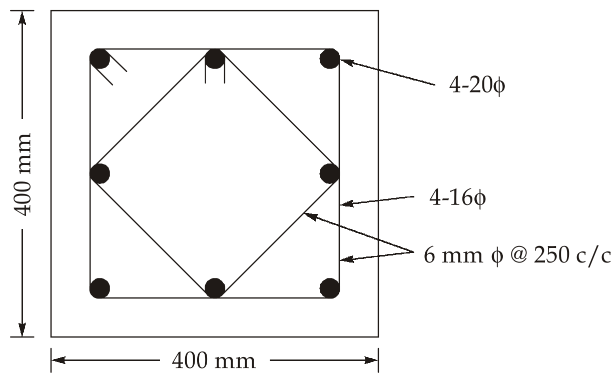
**Step-5:** Lateral ties

Diameter of tie, $\phi_t \geq \begin{cases} \dfrac{\phi_{max}}{4} = \dfrac{20}{4} = 5 \text{ mm} \\ 6 \text{ mm} \end{cases}$

Adopt tie diameter as 6 mm.

Spacing of ties, $S_t \leq \begin{cases} \text{Least lateral dimension of column} = 400 \text{ mm} \\ 16 \times \phi_m = 16 \times 16 = 256 \text{ mm} \\ 300 \text{ mm} \end{cases}$

$\therefore$ Provide 6 mm $\phi$ ties @ 250 mm c/c

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