18CV53

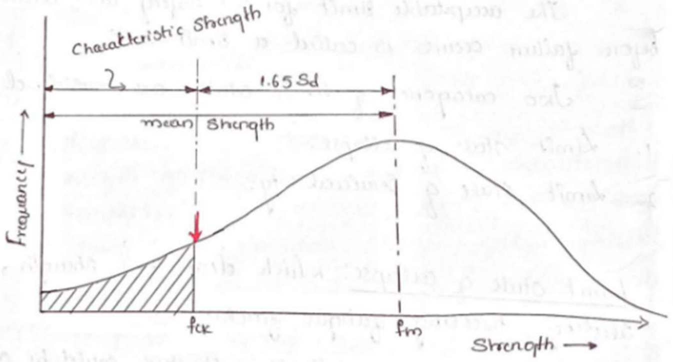
FIFTH SEMESTER B.E. DEGREE EXAMINATION, JAN / FEB 2023

DESIGN OF RC STRUCTURAL ELEMENTS

1) a. Explain characteristic values and design values for strength and load.

Characteristic Strength of materials:

Value of strength of material below which not more than a minimum acceptable percentage of test results are expected to fall. Most of design codes adopted the minimum acceptable percentage as 5% for reinforced concrete structures. This implies that there in only 5% probability or chance of the actual strength being less than the characteristic strength or in other words, the characteristic strength has 95% reliability.



Characteristic Strength = [Mean Strength] - K x [Standard deviation]

$$f_k = f_m - K S_d$$

f_k = Characteristic Strength of Material

f_m = mean Strength K = Constant = 1.65

S_d = Standard deviation for a set of test results.

Characteristic load & Design load.

A Characteristic load is defined as the value of load which has a 95% probability of not being exceeded during the life of structure.

Thus the characteristic value of a particular load can be calculated theoritically. However research for determining actual loading on structures has not yielded adequate data to enable us to compute theoritical values of variations for arriving @ the actual loading on a structure. Code states that since the data are not available to express loads in statistical terms, the loads given in respective code books are assumed as the characteristic loads.

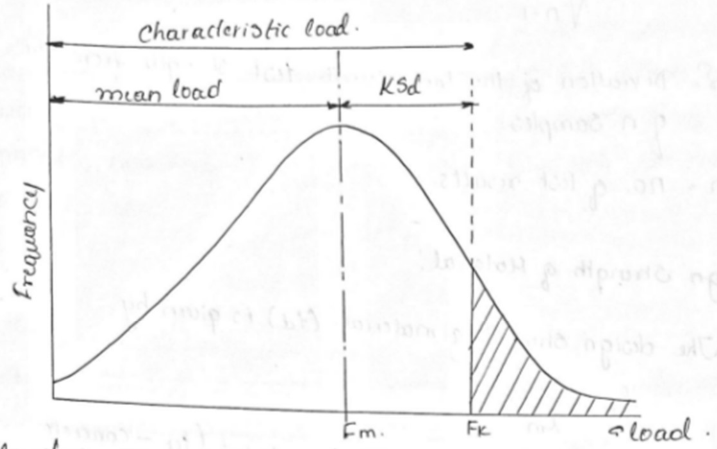
$$F_k = F_m + K S_d$$

F_k = Characteristic load.

F_m = Mean load.

K = Constant = 1.645 ≅ 1.65

S_d = Standard deviation for load.



Design loads: The design load (F_d) is given by.

$F_d = F_k \cdot \lambda_f$. (also Known as Factord load)

F_k = Characteristic load.

λ_f = Partial Safety factor.

b. Differences between working stress method and limit state method.

• Working Stress Method (WSM)

The conceptual basis of WSM is simple. The method basically assumes that the structural material behaves in a linear elastic manner, and the adequate safety can be ensured by suitably restricting the stresses in the material induced by the expected "working load" (Service load) on structure. As specified permissible (allowable) stresses are kept well below. the material strength (i,e in the initial phase of stress strain curve), the assumption of linear elastic behaviour is considered justifiable. The ratio of strength of material to the permissible stress is often referred to as the factor of safety.

The stresses under the applied loads are analysed by applying the methods of 'strength of material'. Such as simple bending theory. In order to apply such methods to a composite material like reinforced concrete, strain compatibility (due to bond) is assumed, whereby the strain in reinforcing steel is assumed to be equal to the adjoining concrete to which it is bonded. Further more, as the stresses in concrete and steel are assumed to be linearly related to there respective strains, it follows that the stress in steel in linearly related to adjoing concrete. by a constant factor (Modular ratio),

The stresses under working load within the permissible stresses are not found realistic by the assumptions made. This may be because of the following reasons.

① Perm effect of creep and shrinkage

② The effect of stress concentration

③ And other secondary ... effects.

All such effects result in significant local increase in re-distribution of calculated stresses.

WSM does not provide realistic measure of actual factor of safety.

LIMIT STATE METHOD [LSM]

An ideal method is the one which takes into account not only the ultimate strength of the structure but also the serviceability and durability requirements. The newly emerging limit state method of design is oriented towards the simultaneous satisfaction of all requirements.

A structure is designed for safety against collapse (for ultimate strength to resist ultimate load) and checked for its serviceability @ working load. The LSM includes consideration of a structure @ both the working and ultimate load level with a view to satisfy the requirements of safety and serviceability.

"The acceptable limit of safety and serviceability requirements, before failure occurs is called Limit State"

c. Explain in detail with sketches of balanced section, under reinforced section, and over reinforced section.

Ultimate Moment of Resistance in terms of Tensile Stress.

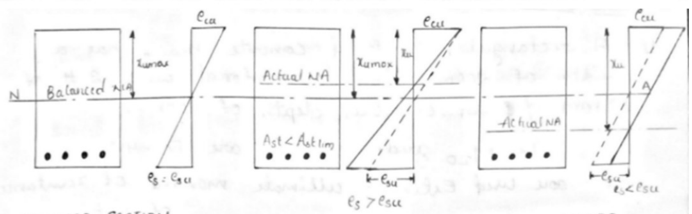
If $\frac{x_u}{d}$ is less than limiting value, calculate moment of resistance by eqn $M_u = 0.87 f_y A_{st} d \left[1 - \frac{f_y A_{st}}{b d}\right]$

Thus, for given section, the actual value of x_u/d (i.e. position of NA) is determined from eq ①. Three cases may arise.

Case 1: x_u/d "equal" to limiting value, $\frac{x_{umax}}{d}$: "BALANCED SECTION"

Case 2: $\frac{x_u}{d}$ "less" than limiting value $\frac{x_{umax}}{d}$: "UNDER-REINFORCED SECTION"

Case 3: $\frac{x_u}{d}$ "More" than limiting value $\frac{x_{umax}}{d}$: "OVER REINFORCED SECTION"



BALANCED SECTION

UNDER REINFORCED SECTION

OVER REINFORCED SECTION.

Balanced Section: The strain in steel and strain in concrete reach their maximum value simultaneously. i.e. $e_c = e_{cu}$ & $e_s = e_{su}$. The % of steel in this section is known as critical or limiting steel percentage ($P_{t\,lim}$). The depth of neutral axis $x_u = x_{umax}$.

Under Reinforced Section: is one in which P_t is less than critical or limiting percentage. Due to this the actual NA is above the balanced NA & $x_u < x_{umax}$. Hence stress in steel required reaches first than concrete.

Beam fails by excess yielding of steel.

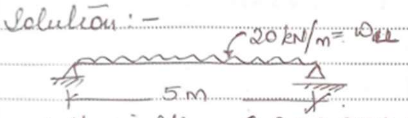
Before beam fails it gives sufficient warning.

Over Reinforced Section: In this type of beam c/s the % of steel is greater than what is required for balanced section. Hence stress in concrete reaches first than steel. Beam fails by crushing of concrete in compression zone. Hence this type of failure is sudden & it won't give warning before it fail.

∴ IS 456 is not permitting over reinforced design.

2) a. **A rectangular simply supported beam of span 5 m is 300 mm x 650 mm in c/s and is reinforced with 3 bars of 20mm on tension side at an effective cover of 50mm. Determine the short term deflection due to an imposed working load of 20 kN/m. Assume grade of concrete M20 and grade of steel Fe415.**

Solution :-



Self weight $= 0.3 \times 0.65 \times 25 = 4.3$ kN/m.

Total udl $= 20 + 4.3 = 24.3$ kN/m.

$M_{max} = \dfrac{wl^2}{8} = \dfrac{24.3 \times 5^2}{8} = 76$ kN-m.

$A_{st} = 3 \times \pi/4 \times 20^2 = 942.48 \text{ mm}^2$

$f_{ck} = 20$ MPa

$f_y = 415$ MPa.

Short term deflection

$$a_i = \frac{5wl^4}{384 E I_{eff}}$$

$$= \frac{5w \cdot l^2 \times l^2}{48 \times 8 \times E \times I_{eff}}$$

$$= \frac{5l^2}{48 E I_{eff}} \times \frac{wl^2}{8} = \frac{5Ml^2}{48 E I_{eff}}$$

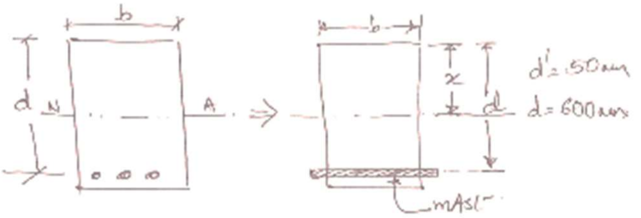
$E_c = 5000 \times \sqrt{f_{ck}} = 5000 \times \sqrt{20}$

$= 22361 \text{ N/mm}^2$

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

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

Modular ratio 'm' $= \dfrac{E_s}{E_c} = \dfrac{2 \times 10^5}{22361} = 8.94$



$m A_{st} \times (d - x) = b \times x \times x/2$

$8.94 \times 942.48 \times (600 - x) = 300 \times \dfrac{x^2}{2}$

$150x^2 + 8425.77x - 5055462.72 = 0$

$x^2 + 56.17x - 33703.1 = 0$

$\therefore \boxed{x = 157.63 \text{ mm}}$

$$I_r = m\,A_{st}(d-x)^2 + \frac{b x^3}{3}$$

$$= 8.94 \times 942.48 \times (600-157.63)^2 + \frac{300 \times (157.63)^3}{3}$$

$$\boxed{I_r = 2.041 \times 10^9\ mm^4}$$

Cracking moment 'M_r' $= \dfrac{f_{cr} \times I_{gr}}{y_t}$

$$f_{cr} = 0.7\sqrt{f_{ck}} = 0.7 \times \sqrt{20} = 3.13\ N/mm^2$$

$$I_{gr} = \frac{300 \times 650^3}{12} = 6.86 \times 10^9\ mm^4$$

$$z = \text{lever arm} = d - \frac{x}{3} = 600 - \frac{157.63}{3}$$

$$= 547.46\ mm.$$

$$y_t = \frac{650}{2} = 325\ mm.$$

$$\therefore M_r = \frac{3.13 \times 6.86 \times 10^9}{325} = 66.067 \times 10^6\ N\text{-}mm$$

$$I_{eff} = \frac{2.041 \times 10^9}{1.2 - \dfrac{66.067 \times 10^6}{76 \times 10^6} \times \dfrac{547.46}{600}\left(1 - \dfrac{157.63}{600}\right) \times \dfrac{300}{300}}$$

$$= \frac{2.041 \times 10^9}{1.2 - 0.585} = 3.32 \times 10^9\ mm^4$$

$I_r < I_{eff} < I_{gr}$ → Hence OK.

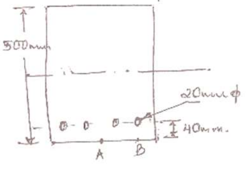
Short term deflection

$$a_i = \frac{5 M l^2}{48 E I_{eff}}$$

$$a_i = \frac{5 \times 76 \times 10^6 \times 5000^2}{48 \times 22361 \times 3.32 \times 10^9}$$

$$\boxed{a_i = 2.666\ mm}$$

b. A reinforced concrete beam of size 250 mm x 500mm is provided with 4 bars of 20mm with an effective cover of 40 mm as shown in figure below. The section has to resist a bending moment of 60 kN-m. Determine the crack width at Point A which is the midpoint of tension edge and at point B, which is on tension edge just below bar M20 and Fe415 steel used.



$M =$

$M = 60\ kN\cdot m$

$A_{st} = 4 \times \frac{\pi}{4} \times 20^2 = 1256.64\ mm^2$

$E_c = 5000\sqrt{f_{ck}} = 5000 \times \sqrt{20} = 22360\ N/mm^2$

$E_{ce} = \frac{E_c}{2} = 11180\ N/mm^2$

$m = \frac{E_s}{E_{ce}} = \frac{2 \times 10^5}{11180} = 17.89$

$\therefore$ To determine depth of N.A. for an elastic section

$b \cdot x \cdot \frac{x}{2} = m \cdot A_{st}\,(d - x).$

$250 \times \frac{x^2}{2} = 17.89 \times 1256.64 \times (500 - 40 - x)$

$125x^2 = 10341393.2 - 22481.29x$

$x^2 + 179.85x - 82731.15 = 0.$

$\boxed{x = 211.435\ mm}$

To find out the moment of Inertia (I_c) of the cracked section

$I_c = \frac{bx^3}{3} + mA_{st}(d - x)^2$

$I_c = \frac{250 \times 211.435}{3} + 17.89 \times 1256.64 \times (460 - 211.435)^2$

$\boxed{I_c = 1.389 \times 10^9\ mm^4}$

We know that

$\frac{M}{I} = \frac{f}{y} = \frac{\epsilon E}{y}$

$\therefore \epsilon = \frac{M \cdot y}{EI}$

Strain at any distance x_1 from the N.A is given by

$\epsilon_1 = \frac{M}{E_{ce}} \frac{x_1}{I_c}.$

bottom most fibre $x_1 = 500 - x$

$= 500 - 211.435$

$= 288.565\ mm$

$\therefore$ strain $\epsilon_1 = \frac{60 \times 10^6}{11180 \times 1.389 \times 10^9}$

$\therefore \epsilon_1 = 3.864 \times 10^{-6}.$

$\epsilon_m = \epsilon_1 - \frac{b(D - x)(a - x)}{3E_s A_s (d - x)}$

$= 3.864 \times 10^{-6} - \frac{250(500 - 211.435)(500 - 211.435)}{3 \times 2 \times 10^5 \times 1256.64 \times (460 - 211.435)}$

$= \cancel{3.864 \times 10^{-6}} - \cancel{9.568 \times 10^{-5}}$

$= 0.00011$

Crack width at point 'A' located at the midpoint

$a_{cr_A} = \sqrt{40^2 + 18.89^2}$

$= 44.23\ mm$

$\epsilon_m = 0.00011$

$w_A = \frac{3a_{cr}\epsilon_m}{1 + \frac{2(a_{cr} - C_{min})}{h - x}}$

$= \frac{3 \times 44.23 \times 0.00011}{1 + \frac{2(44.23 - 30)}{500 - 211.435}} = 0.0133\ mm$

Crack width at point 'B' located at the bottom of bar

$a_{cr_B} = 40\ mm$

$w_B = \frac{3 \times 40 \times 0.00011}{1 + \frac{2(40 - 30)}{500 - 211.435}} = \frac{0.0132}{1.069} = 0.0123\ mm$

3) **Determine the moment of resistance of T section having the following section properties. Width Of flange = 2500 mm, Depth of flange = 150 mm, Width of rib = 300mm, effective depth = 800 mm, Area of steel = 8 bars of 25 mm diameter. Use M20 and Fe415 HYSD bar. Similar solution.**

Effective width of flange (b_f)

$$b_f = \frac{l_o}{\frac{l_o}{b} + 4} + b_w$$

$l_o = l = 3600mm$

$$b_f = \frac{3600}{\frac{3600}{2400} + 4} + 300 = 954.54 mm < \text{actual width} \; (b = 2400mm)$$

Depth of neutral axis (x_u)

Assuming neutral axis lies in the flange.

$$x_u = \frac{0.87 f_y A_{st}}{0.36 f_{ck} \cdot b_f} = \frac{0.87 \times 415 \times 3041}{0.36 \times 20 \times 954.54}$$

$= 159.75mm > D_f$

Hence, our assumption was wrong.

The value of x_u is slightly more than D_f. Therefore, it may be the case:-

$D_f > \frac{3}{7} x_u$ or $\frac{D_f}{x_u} > 0.43$

The depth of neutral axis can be found by using

$$0.36 f_{ck} \cdot b_w x_u + 0.45 f_{ck} y_f (b_f - b_w) = 0.87 f_y \cdot A_{st}$$

$$0.36 \times 20 \times 300 x_u + 0.45 \times 20 \times y_f (954.54 - 300) = 0.87 \times 415 \times 3041$$

$2160 x_u + 5890.94 y_f = 1097958$

$y_f = 0.15 x_u + 0.65 D_f$

$= 0.15 x_u + 0.65 \times 120$

$= 0.15 x_u + 78$

$\therefore 2160 x_u + 5890.94 (0.15 x_u + 78) = 1097953$

$3043.64 x_u = 638459.68$

$x_u = 209.76 mm$

$\frac{3}{7} x_u = \frac{3}{7} \times 209.76 = 90.2mm < D_f$.

Hence our assumption was correct.

$x_{umax} = 0.48d$

$= 278.4mm$

$x_{umax} > x_u$. Hence, the section is under reinforced.

$y_f = 0.15 \times 209.76 + 0.65 \times 120$

$= 109.46mm < D_f$ hence OK.

Moment of resistance of the section (M_u).

$$M_u = 0.36 \frac{x_u}{d}\left(1 - 0.42 \frac{x_u}{d}\right) f_{ck} b_w d^2 + (0.45 f_{ck} (b_f - b_w) y_f \left(d - \frac{y_f}{2}\right)$$

$$= 0.36 \times \frac{209.76}{580}\left(1 - \frac{0.42 \times 209.76}{580}\right) \times 20 \times 300 \times 580^2 + 0.45 \times 20 (954.54 - 300) \times 109.46 \left(580 - \frac{109.46}{2}\right)$$

$= 562500695$ N-mm

$= 562.5$ kN-m

OR

4) **A doubly reinforced concrete beam having a rectangular section 250 mm wide and 540 mm overall depth is reinforced with 2 bars of 12mm diameter in the compression side and 4 bars of 20mm diameter in the tension side. The effective cover to bars is 40mm. Using M20 grade concrete and Fe415 HYSD bars, estimate the flexural strength of the section using IS 456 – 2000 code recommendations. Similar solution.**

$b = 250mm$, $D = 500mm$, $d' = 40mm$, $d = 500 - 40 = 460mm$

$f_{ck} = 20 N/mm^2$ $f_y = 250 N/mm^2$

$A_{st} = 6 \times \frac{\pi}{4} \times 20^2 = 1885 mm^2$, $A_{sc} = 2 \times \frac{\pi}{4} \times 20^2 = 628 mm^2$

Assuming compression as well as tensile steel yielded.

$f_{sc} = f_{st} = 0.87 f_y$.

$0.36 f_{ck} x_u b + 0.87 f_y A_{sc} = 0.87 f_y A_{st}$

$0.36 \times 20 \times x_u \times 250 + 0.87 \times 250 \times 628 = 0.87 \times 250 \times 1885$

$\therefore x_u = 151.89 mm$

$x_{ulim} = 0.53d = 0.53 \times 460 = 243.8mm$.

$x_u < x_{ulim}$ or x_{umax}. $\therefore$ Tensile steel has yielded.

Strain in compression steel.

$$\varepsilon_{sc} = 0.0035\left(1 - \frac{d'}{x_u}\right) = 0.0035 \times \left(1 - \frac{40}{151.89}\right)$$

$= 2.578 \times 10^{-3}$

Minimum strain at which yielding starts in compression steel.

$$\varepsilon_y = 0.87 \frac{f_y}{E_s} + 0.002 = 1.0875 \times 10^{-3}$$

$\therefore \varepsilon_{sc} > \varepsilon_y$

$\therefore$ Assumption that stress in compression steel is $0.87 f_y$ is correct. $\therefore x_u = 151.89mm$

$M_u = 0.36 f_{ck} x_u b (d - 0.42 x_u) + f_{sc} A_{sc} (d - d')$

$= 165.69$ kN-m

5) **Design a rectangular beam of section 230 mm x 600 mm of effective span 6 m effective cover for reinforcement should be kept as 50mm. Imposed load on the beam is 40 kN/m. Use M20 concrete and Fe415 steel. Similar solution.**

$b = 250$ mm $\quad f_{ck} = 20$ N/mm^2

$D = 500$ mm $\quad f_y = 415$ N/mm^2

$d = 450$ mm $\quad E_s = 2\times10^5$ N/mm^2

$d' = 50$ mm

$l_{eff} = 5$ m $\quad w = 40$ kN/m $\quad w_u = 40\times1.5 = 60$ kN/m

Step 1: Compare M_u & M_{ulimit}

$$M_u = \frac{W_u l_{eff}^2}{8} = \frac{60\times5^2}{8} = 187.5 \text{ kN-m}$$

$$V_u = \frac{W_u l_{eff}}{2} = 150 \text{ kN}$$

Step 2: Determination of M_{ulimit}

$$M_{ulimit} = 0.138 f_{ck} b d^2$$

$$= 0.138\times20\times250\times450^2$$

$$= 140 \text{ kN-m}$$

Compare M_u & M_{ulimit}

$M_u > M_{ulimit}$

$(M_u - M_{ulim}) = 187.5 - 140 = 47.5$ kN-m

$f_{sc} = \varepsilon_{sc}\times E_s$

where, $f_{sc} = \left\{\frac{0.0035(x_{umax} - d')}{x_{umax}}\right\}\times E$

$$f_{sc} = \left\{\frac{0.0035(0.48\times450 - 50)}{(0.48\times450)}\right\}\times 2\times10^5$$

$$= 538 \text{ N/mm}^2$$

But $f_{sc} \not> 0.87 f_y = (0.87\times415) = 361$ N/mm^2

Therefore $f_{sc} = 361$ N/mm^2

Steel $A_{sc} = \left[\frac{M_u - M_{ulim}}{f_{sc}(d-d')}\right] = \left[\frac{47.5\times10^6}{361\times400}\right] = 329$ mm^2

Provide 2# of 16mm ϕ ($A_{sc} = 402$ mm^2)

$$A_{st_2} = \left(\frac{A_{sc} f_{sc}}{0.87 f_y}\right) = \left(\frac{329\times361}{0.87\times415}\right) = 329 \text{ mm}^2$$

$$A_{st_1} = \left[\frac{0.36 f_{ck}\, b\, x_{ulim}}{0.87 f_y}\right]$$

$$= \left[\frac{0.36\times20\times250\times0.48\times450}{0.87\times415}\right]$$

$$A_{st_1} = 1077 \text{ mm}^2$$

Total tension reinforcement, $A_{st} = A_{st_1} + A_{st_2}$

$= 1077 + 329$

$= 1406$ mm^2

Provide 3# of 25mm ϕ

$A_{st} = 1473$ mm^2

Step 3: Check for Shear

$$\tau_v = \frac{V_u}{bd} = \frac{150\times10^3}{250\times450} = 1.33 \text{ N/mm}^2$$

$$P_t = \frac{100 A_{st}}{bd} = \frac{100\times1473}{250\times450} = 1.3$$

Referring to table 19

$\tau_c = 0.66$ N/mm^2 by interpolation

Comparing τ_c & τ_v

$\tau_v > \tau_c$ Hence provide design shear reinforcement

$V_{us} = V_u - \tau_c bd$

$= 150 - (0.66\times250\times450)\times10^{-3}$

$= 73.5$ kN

Using 2LVs of 8mm ϕ

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

$$= \frac{0.87\times415\times100\times450}{73.15\times10^3}$$

$= 221$ mm

Maximum spacing is $0.75d = 0.75\times450 = 337.5$ mm

300 mm

220 mm

∴ Provide 2LVs of 8mm ϕ @ 220 mm c/c.

Step 4: Check for deflection control.

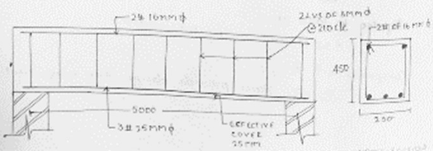
$$(l/d)_{all} = (l/d)_{basic}\times K_t\times K_c\times K_f$$

$$= 7\times0.93\times1.1\times1$$

$$(l/d)_{allo} = 20.46 > (l/d)_{act} = \left(\frac{5000}{450}\right) = 11.1$$

Hence deflection control is satisfied

Step 5: Detailing.



6) Design a simply supported beam of span 5m carries a live load of 12 kN/m. Use M20 grade of concrete and Fe415 steel. Similar solution

Step 1 (i) Fixing up the depth of Section

Taking $\frac{L}{d} = 20$ [Pn 37, Cl N 23.2.1] for SSB

$$d = \frac{L}{20} = \frac{5}{20} = 0.25\,m = 250\,mm$$

Providing effective cover of 25 mm, $D = d' + d$

$$= 25 + 250$$
$$= 275\,mm$$

Assuming $b = 230$

$d = 250$

(ii) Check for lateral Stability. [PN-39, Cl NO. 23.3]

Allowable $L = 60b$ or $\frac{250b^2}{d}$

i.e $= 60 \times 230$ or $\frac{250 \times 230^2}{250}$

$= 13800\,mm$ (or) $= 52.9\,m$

$= 13.8\,m$

Given L is lesser of above two Hence OK

(iii) Effective Span for SSB [PN 34, Cl NO 22.2]

l_{eff_1} = Clear span + Effective depth $= 5 + 0.25 = 5.25\,m$

l_{eff_2} = ½ thickness of Sup + L + ½ T.S $= \frac{0.23}{2} + 5 + \frac{0.23}{2} = 5.23\,m$

Lesser of above two values should be taken

i.e $l_{eff} = 5.23\,m$

- **Step 2: Load calculation**

Considering 1m length of beam

a. Dead load $= (0.23 \times 0.275 \times 25) = 1.58\,kN/m$

b. Live load $= 25\,kN/m$

Total working load $26.58\,kN/m$

Ultimate load = or Factored load $= W_u = 26.58 \times 1.5$

$= 39.87\,kN/m$

$W_u \simeq 40\,kN/m$

Step 3: BM & SF calculation

$$M_u = \frac{W_u l_{eff}^2}{8} = \frac{40 \times 5.23^2}{8} = 136.76\,kN\text{-}m$$

$$V_u = \frac{W_u l_{eff}}{2} = \frac{40 \times 5.23}{2} = 104.6\,kN$$

- **Step 4: Check for depth on Bending moment consideration**

Assuming Section to be balanced $M_u = M_{ulim}$

i.e for Fe 415 $M_{ulim} = 0.138 f_{ck} bd^2$

$$d = \sqrt{\frac{M_{ulim}}{0.138 f_{ck} b}}$$

$$d = \sqrt{\frac{136.76 \times 10^6}{0.138 \times 20 \times 230}} = 464.21\,mm$$

$$d_{req} = 465\,mm$$

$d_{prov} = 250\,mm$ Hence Revise the Section

taking $d = 500\,mm$

$b = 230\,mm$

$D = 525\,mm$

* Loads

Dead load = 0.23 × 0.525 × 25 = 2.875

Live load = 25

Total working load = 27.875 kN/m

Factored load = 27.87 × 1.5 = 41.8 ≅ 42 kN/m

* $M_u = \frac{42 \times 5.23^2}{8} = 143.6$ kN-m

* $V_u = \frac{42 \times 5.23}{2} = 109.83$ kN.

* Check for depth.

$M_u = M_{ulim} = 143.6 \times 10^6 = 0.138 \times f_{ck} b d^2$

$d = \sqrt{\frac{143.6 \times 10^6}{0.138 \times f_{ck} \times b}}$

$d = \sqrt{\frac{143.6 \times 10^6}{0.138 \times 20 \times 230}}$

$d_{req} = 475.68$ mm

We decided to provide 500mm depth, d_{req} for the Moment = 475mm

Hence OK.

∴ We shall continue with d = 500mm   D = 525mm

* Check Section is Under Reinforced

Actual Moment acting • $M_u = 143.6$ kN·m.

• $M_{ulimit} = 0.138 f_{ck} b d^2$

$= 0.138 \times 20 \times 230 \times 500^2$

$= 158.7 \times 10^6$ N-mm

i.e $M_u < M_{ulimit}$

Hence Section is Under Reinforced.

Comparing τ_c & τ_v

i.e $\tau_c < \tau_v$

Provide Design Shear Reinforcement. - Vertical Stirrups.

∴ $V_{us} = \frac{0.87 f_y A_{sv} d}{S_v}$

$V_{us} = V_u - \tau_c b d.$

$= 105 \times 10^3 - 0.58 \times 230 \times 500$

$= 38.3$ kN.

$A_{sv} = \frac{\pi \times 8^2}{4} \times 2 = 100 \text{ mm}^2$

$S_v = \frac{0.87 \times 415 \times 100 \times 500}{38.3 \times 10^3}$

$S_v = 471.3$ mm.

* Check for Maximum Spacing.

1. 0.75D = 0.75 × 500 =
2. 300 mm
3. Calculated value above.

Hence provide 2LVS of 8mm ϕ @ 300mm c/c.

7) A hall has clear dimension 3 m x 9m with wall thickness 230mm. The live load on the slab is 3 kN/m^2 and a finishing load of 1 kN/m^2 may be assumed. Use M20 grade concrete and Fe415 grade steel. Design the slab.

Step1! Deciding type of Slab:-

① $\frac{l_y}{l_x} = \frac{L_y}{l_x} = \frac{9700}{3600} = 2.6 \simeq$ One way Slab.

Hence design it as one way slab.

② Effective depth = d =

$\left(\frac{L}{d}\right) = \frac{3600}{26 \times 1.2} = 115.3\text{mm} \simeq 120\text{mm}$

i.e d = 120mm, d' assumed as = 15, D = 135mm

Step 2: Load Calculation.

Self wt of slab = 0.135 X 1 X 25 = 3.375 KN/m

Live load = 3 X 1 = 3 KN/m

Floor finish = 1 X 1 = 1 KN/m

Partition load = 1 X 1 = 1 KN/m.

Total working 8.375 KN/m.

Ultimate working = 8.375 X 1.5 = 12.56 KN/m.