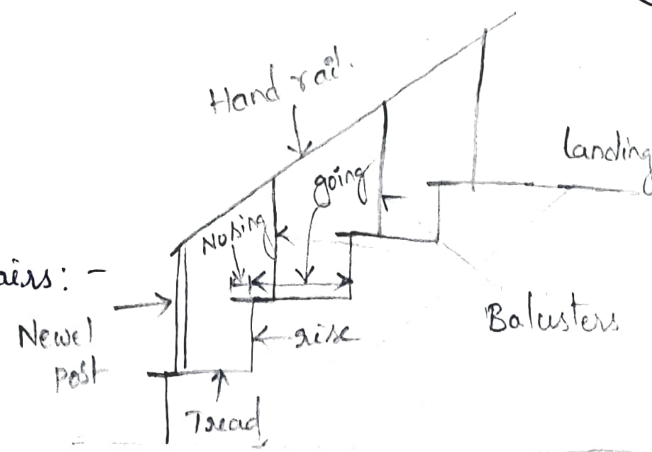
UNIT - VI

Design of Stair cases :-



Requirements and standards of good stairs :-

* The following are the standards that have been recommended by NBC (National building code) muncipal rules.

(i) Minimum width :-

For residential buildings - 1m

For Hotel buildings - 1.5m

For assembly buildings like auditorium, theatres - 1.5m

For educational buildings - 1.5m

Institutional buildings - 2m (Hospitals)

All other buildings - 1.5m

(ii) Minimum Tread :-

The minimum width of tread without nosing should be 250mm for residential building and for other it should be 300mm.

(iii) Minimum rise :-

The Maximum height of rise should be 190mm for residential building and 150mm for other buildings. The number of steps in a flight should be limited to 12.

To have easy climbing (or) descending, the following relationship between going and rise are adopted.

(a) Going in mm + 2 x rise = 600

(b) Going in mm x Rise in mm = 40,000 to 41 000

(iv) Head room :-

The minimum head room under the landing in any stair case should be 2.2m.

(v) Hand rails :-

The hand rails should have a minimum height of 0.9m from the centre of tread.

(vi) Landing :-

The width of landing should be same as the width of stair.

Types of stairs :-

1. Straight flight stair case
2. Quarter turned stair case
3. Dog legged stair case
4. open well stair case.
5. spiral stair case.
6. Heli Coidal stair case
7. Free standing stair case.

Requirements of Good stair :-

(i) No. of risers = $\frac{\text{Total height of floor}}{\text{Height of riser}}$

(ii) No. of treads in a flight = No. of risers − 1

(iii) Rise + Going = 40 to 45 cm.

(iv) 2 × Rise + Going = 60 to 64 cm.

(v) Rise × Going = 400 to 426 cm^2

(vi) Rise = 140 to 160mm

(vii) Tread = 250 to 300 mm.

(viii) The width of stair should be sufficient for two persons to pass on it. The minimum width of stair is 80cm.

(ix) Inclination i.e pitch of stair = 30 to 45°

(x) The clear distance b/n tread and soffit of a flight immediate above is called as head room which should not be less than 2.1 m.

(xi) A landing is provided after 12 to 15 steps, width of landing should not be less than width of stair.

General Notes on design of stairs :-

(1) Live load on stairs:-

According to IS: 875- 1964 (code of Practice for structural safety of buildings) gives the loads for stair cases.

For stairs in residential buildings, office buildings, hospital buildings, hostels etc.. where there is no possibility of over crowding, the live load may be taken as 3000 N/m^2 subjected to a minimum of 1300 N concentrated load at the unsupported end of each step.

For other public buildings reable to be over crowded, the live load may be taken as 5000 N/mm^2.

(2) Effective span of slabs :-

Stair slab may be divided into two categories depending upon the direction in which the stair slab spans.

(i) The stair slab spanning horizantally

(ii) The stair slab spanning longitudinally.

1) Design a Dog-legged Stair for a building in which the vertical distance between the floors is 3.6m. The stair hall measured 2.5m x 5m, the live load may be taken as 2.5 kN/m^2. Use M_{20} and Fe_{415} materials.

Sol:- Given data :-

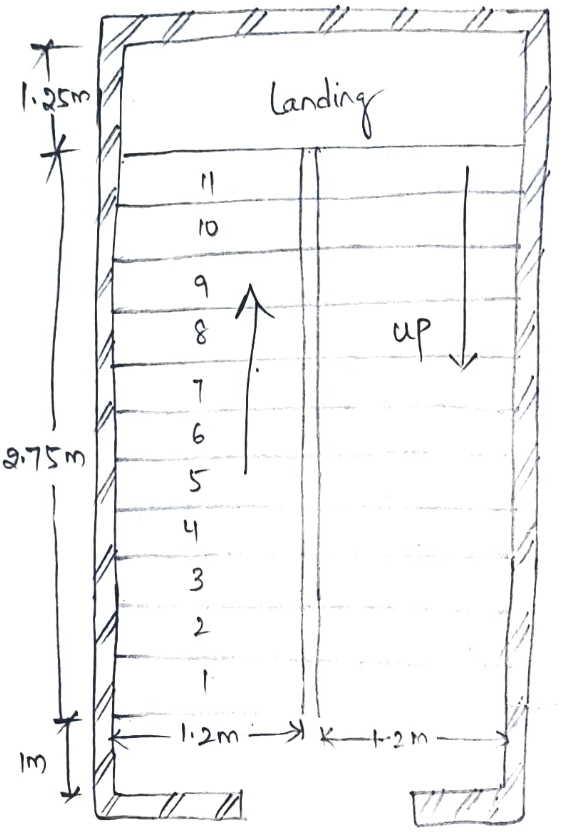
Height of the room = 3.6m

Size of the room = 2.5 x 5m.

live load = 2.5 kN/m^2

f_{ck} = 20 N/mm^2

f_y = 415 N/mm^2



step-1:- General arrangement:-

The figure shows the plan of stair hall.

Let the rise be 150mm and tread be 250mm.

Let us keep width of each flight = 1.2m

Height of each flight = $\frac{3.6}{2}$ = 1.8 m

No. of risers required = $\frac{1.8}{0.15}$ = 12 risers in each flight

No. of treads = No. of risers − 1 = 12 − 1 = 11 Treads in each flight.

space occupied by treads = 11 x 250 = 2750mm.

Keep width of landing = 1.25m

Hence space left for passage = 5 − 1.25 − 2.75 = 1m

Step-2:- loading on each flight :-

The landing slab is assumed to span in the same direction as the stairs and is considered as acting together to form a single slab. Let the bearing of the landing slab in the wall be 160mm. Then effective span $= \frac{0.16}{2} + 1.25 + 2.75$

$= 4.08m \simeq 4.1m.$

Assume overall depth of waist slab = 150mm.

Assume clear cover = 15mm, use 12 mm dia bars.

Effective depth, $d = 150 - 15 - 12/2 = 129$ mm.

self weight of waist slab 'w' on slope = $0.15 \times 1 \times 25 = 3.75 \ KN/m^2$

Dead weight on horizontal area, $w_1 = \frac{w \times \sqrt{R^2+T^2}}{T}$

$= \frac{3.75 \times \sqrt{0.15^2+0.25^2}}{0.25} = 4.372 \ KN/m^2$

Dead weight of step is given by $w_2 = R/2 \times 1 \times 25$

$= \frac{0.15}{2} \times 1 \times 25 = 1.875 \ KN/m^2.$

Assume floor finish = $0.1 \ KN/m^2$

live load = $2.5 \ KN/m^2$

$\therefore$ Total load = $w_1 + w_2 + 0.1 + 2.5 = 4.372 + 1.875 + 2.6 = 8.847 \ KN/m^2$

Factored load = $8.847 \times 1.5 = 13.27 \ KN/m^2$

The value of w on the landing portion will be $8.847 - 1.875$
$= 6.972 \ KN/m^2$
($\because$ weight of steps will not be considered)

Step-3:- Depth of slab from bending moment consideration :-

$$M_u = \frac{W_u l^2}{8} = \frac{13.27 \times 4.1^2}{8} = 27.885 \text{ KNm.}$$

shear force, $V_u = \frac{wl}{2} = \frac{13.27 \times 4.1}{2} = 27.204$ KN.

For a balanced design, $M_u = 0.138 f_{ck} b d^2$

$$27.885 \times 10^6 = 0.138 \times 20 \times 1000 \times d^2$$

$$d = 100.51 \text{ mm.} < \cancel{150}\ 129 \text{ mm.}$$

Hence safe.

step-4:- Area of steel (Per meter width of stair) :-

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

$$27.885 \times 10^6 = 0.87 \times 415 \times A_{st} \times 129 \left(1 - \frac{415 A_{st}}{20 \times 1000 \times 129}\right)$$

$$A_{st} = 671.14 \text{ mm}^2/\text{m}$$

A_{st} per 1.2m width $= 1.2 \times 671.14 = 805.37 \text{ mm}^2$

use 12mm dia bars, No. of bars $= \frac{805.37}{\pi/4 \times 12^2} = 7.06 \approx 8$ bars

spacing of bars $= \frac{1200}{8} = 150$mm.

Actual Ast Provided $= \frac{\pi/4 \times 12^2 \times 8}{1.2} = 754.08 \text{ mm}^2$

Distribution reinforcement = 0.12% of Gross c/s area

$$= \frac{0.12}{100} \times 150 \times 1000 = 180 \text{ mm}^2$$

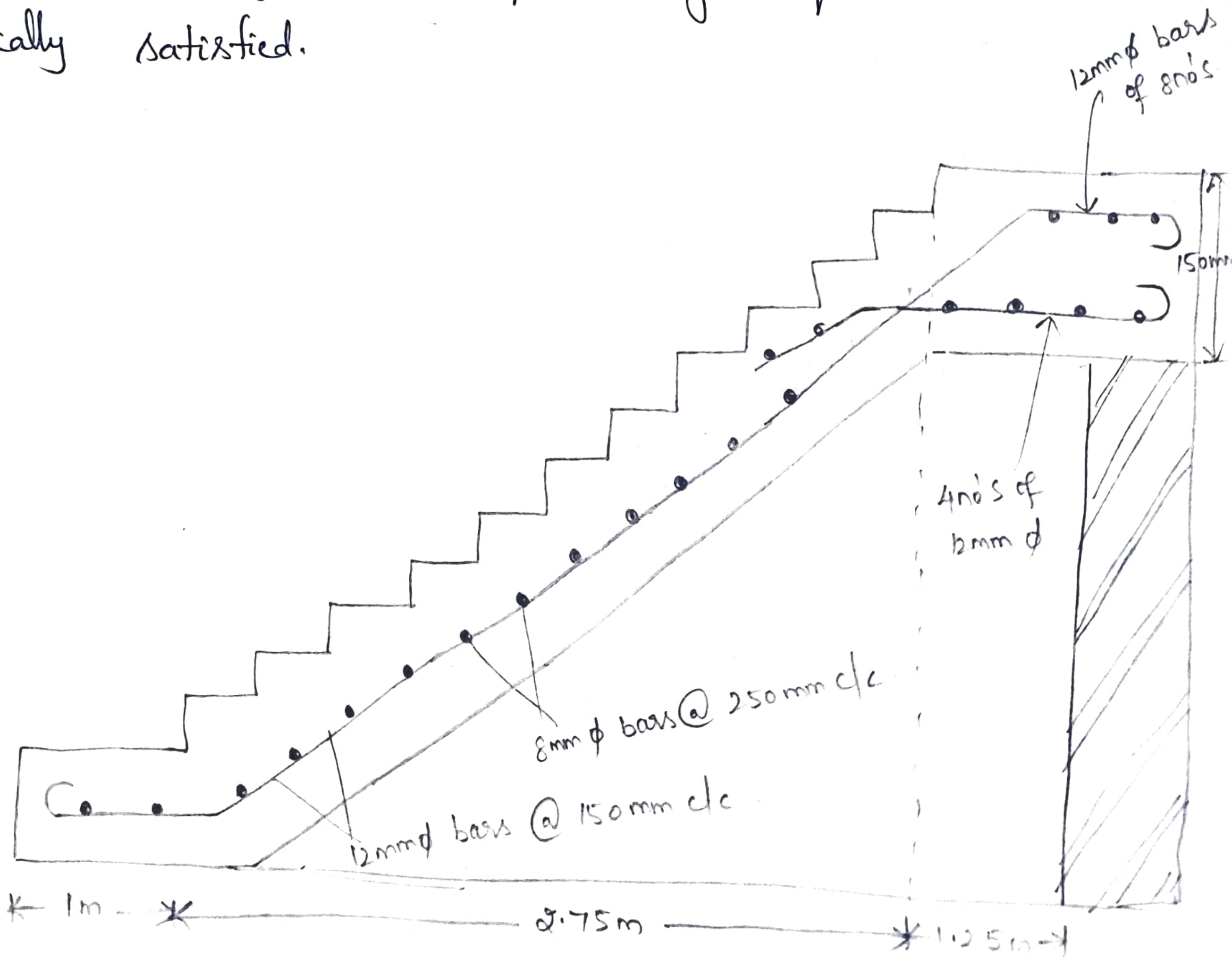
Use 8mm diameter bars, spacing $= \frac{\pi/4 \times 8^2}{180} \times 1000 = 279.3$ mm

Provide 8mm dia bars @ 250 mm c/c.

The main reinforcement should be bend to follow the bottom profile of the stair. However if this pattern is followed near the landing an angle will be formed in the bar. The length of each type of bar on either side of crossing should be atleast equal to

$$L_d = \cancel{54.4}\,47 \times 12 = \cancel{652.8}\,564 \simeq 570 \text{ mm}.$$

Since all the bars of tension reinforcement are taken into supports, the anchorage and development length requirements will be automatically satisfied.



Reinforcement Details.

Serviceability requirements of Reinforced Concrete members :-

Reinforced Concrete members should be designed to confirm to the limit state of strength and serviceability.

(i) The member should not undergo excessive deformation under service loads. This limit state is refered as limit state of deflection.

(ii) The width of cracks developed on the surface of reinforced concrete members under service loads should be limited to the values prescribed in the Code of Practice. This limit state is refered as limit state of cracking.

* Deflection in R.C members may be divided into the following categories.

(1) short term deflection
(2) Long term deflection.

(1) short term deflection :-

* Short term deflection is immediate (or) instantaneous (or) elastic deflection due to the permanant imposed loads under service Condition. This type of deflection leads to the following effects.

(i) Excessive deflection creates feeling of lack of safety.

(ii) It effects the geometry and shape hence the appearance of the structure reduces.

(iii) It leads to misalignment of sensitive ~~[illegible]~~ Machinery and hence effect Smoother functioning and performance.

(2) Long term deflection:-

* This deflection is caused due to creep, shrinkage, under sustained loads and additional short term elastic deflections due to temporary live loads. This type of deflections leads to following effects.

(i) It leads to objectionable cracks, in the walls, floors and roof slabs creating leakage problems.

(ii) It creates problems of poor drianage and damping of roofs.

Total Deflection:-

Total deflection = short term deflection + Long term deflection.

Computation of short term deflection:-

* The short term deflection may be calculated by working stress method.

(1) Modular ratio $K = \dfrac{280}{3\sigma_{cbc}}$

σ_{cbc} = Permissible compressive strength of Concrete

For M_{15}, $\sigma_{cbc} = 5\ N/mm^2$, For Fe_{250}, $\sigma_{st} = 140\ N/mm^2$

For M_{20}, $\sigma_{cbc} = 7\ N/mm^2$, For Fe_{415}, $\sigma_{st} = 230\ N/mm^2$

σ_{st} = Permissible tensile stress of steel.

(2) Singly reinforced section:-

$$\frac{b\,n_a^2}{2} = m\,A_{st}\,(d-n_a)$$

Moment of Inertia of a cracked section, $I_R = \dfrac{b\,n_a^3}{3} + m\,A_{st}\,(d-n_a)^2$

3) Doubly Reinforced Section

$$\frac{b n_a^2}{2} + (m-1) A_{sc} (n_a - d_c) = m A_{st} (d - n_a)$$

$$I_r = \frac{b n_a^3}{3} + (m-1) A_{sc} (n_a - d_c)^2 + m A_{st} (d - n_a)^2$$

Computation of Long term deflection :-

1. Deflection due to shrinkage :

The property of reduction in volume during the process of drying and hardening is called shrinkage.

2. Deflection due to creep :

Creep is the plastic deformation under sustained loads and its effect to increase the strain in concrete at constant stress.

Total deflection :-

* This is sum of short term elastic deflection due to permenant loads and long term deflection due to shrinkage and creep and short term deflection due to temporary and live loads.

$$a_T = a_{i(perm)} + a_{cc\,(perm)} + a_{cs} + a_{i\,(Temp)}$$

where, $a_{i\,perm}$ = Short term initial elastic deflection due to permenant loads using short term young's modulus of concrete (E_c)

$a_{cc\,perm}$ = creep deflection due to permenant loads

$a_{icc\,perm}$ = Initial short term deflection + creep deflection due to permenant load.

a_{cs} = Deflection due to shrinkage.

$a_{i\,temp}$ = Short term deflections due to temporary (or) live loads using young's modulus of Concrete.

Deflection after erection of Partition walls :-

* This is the deflection which causes cracks in partition walls and finishes.
* It includes deflection due to shrinkage, creep and instantaneous elastic deflections due to temporary live loads.

$$a_p = a_{cc} + a_{cs} + a_{i\,perm}$$

a_{cs} = deflection due to part of shrinkage which occurs after the application of loads shall be taken normally 50% of shrinkage strain (0.0003) occurs during the first month and about 75% in 6 months.

IMP

Measures for Control of deflection :-

* If the actual deflection exceeds the permissible deflections, the following measures may be adopted for controlling.

(i) Increase the c/s dimensions preferably the depth.

(ii) Use higher grade of Concrete and lower grade of steel.

(iii) Provide an initial camber of $1/250$ to $1/400$

(iv) Delay in erection of finishes and partitions as long as possible.

(v) Take ~~ad~~ use of flanged ~~section~~ action of the beam, if it is not in the design of reinforcement.

Limit state of Cracking :-

* Limit state method of design considers cracking as one of the important limit states of serviceability, the attainment of which makes the structure unfit.

* Cracks in the R.C members must be controlled not only for easthetic reasons but more importantly for durability and particularly for corrosion protection of reinforcement.

* Crack width is most controlled by ensuring good quality of concrete, proper detailing of shrinkage and proper construction procedure.

Imp

Causes of Cracking :-

* Cracking in concrete generally occurs as a result of

(i) volumetric change including drying shrinkage, creep under sustained loads, ~~and~~ thermal strains and agressive environmental effects.

(ii) Direct and flexural stresses due to bending and shear.

(iii) Internal (or) external strains due to continuity and differential movement of support and adjacent structural members.

(iv) Stress concentration due to curtailement and splicing of bars.

(v) Corrosion of reinforcement.

Adverse effects of cracking :-

(i) It changes the appearance of the exposed surface.

(ii) It creates a feeling of lack of safety when presenting large proportions in walls and beams.

(iii) It reduces the imperviousness creates leakage problem. in floors and tank walls rendering them unserviceable.

(iv) It leads to corrosion of steel and hence reduces the strength and durability of structure.

(v) It reduces the thickness of the members and stiffness of the members and hence increases the deflections.

(vi) It creates ~~to~~ lot of maintainance problems.

Imp

Control of cracking :-

(i) Increasing the grade of Concrete

(ii) use more number of smaller diameter bars.

(iii) Limit the tensile stresses in reinforcement

(iv) Maintaining, thourough controlling and capable supervision during mixing, placing and curing of Concrete.

Design surface crack width 'W_{cr}' :- (Annex F, Pg 95)

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

where, a_{cr} = Distance from the point considered to the surface of the nearest longitudinal bar.

C_{min} = Minimum Cover to the longitudinal bar

ϵ_m = Average steel strain at the level considered.

h = Overall depth of the member.

x = Depth of neutral axis.

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

where, A_s = Area of tension reinforcement

b = width of the section at the centroid of tension steel

ϵ_1 = strain at the level considered, calculated, ignoring the stiffening of the concrete in tension zone.

a = Distance from the compression face to the point at which the crack width is being calculated and

d = effective depth.

$$\epsilon_1 = \frac{f_s}{E_s} \frac{(h-x)}{(d-x)}$$

$$f_s = m \times \frac{M_g y}{I_r}$$

Codal crack width limits :-

* As per IS : 456, the crack width limit of 0.3 mm for the surface crack in R.C member.
* This limit is adequate for the purpose of durability.
* when the structural member is completely protected against aggressive environmental conditions.
* Usually a nominal cover of 25 mm, the limiting crack width is 0.1 mm.
* For structures exposed to moderate environments, the limiting crack width may be taken as 0.2 mm.

Detailing Rules :-

(i) Maximum and minimum spacing of bars (cl 26.3.2 and 26.3.3, Pg 45 & 46)

(ii) Maximum and minimum area of steel (cl 26.5.1.1 and 26.5.1.2, Pg 47)

(iii) Curtailement of reinforcement (cl 26.2.3.2, Pg 44)

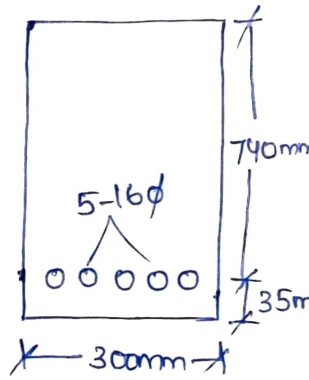
(iv) Anchorage of reinforcement (cl 26.2.2, Pg 43)

(v) splicing of reinforcement (cl 26.2.5, Pg 44)

(vi) cover to main reinforcement (cl 26.4, Pg 46)

1) A simply supported beam has been designed for 6m clear span to carry an U.D.L of 20 kN/m. The dimensions are shown in figure. Use M_{20} and Fe_{415} materials are used. Compute the immediate and longterm deflection check the adequacy of the section as per IS Code.

Sol:- <u>Given data</u> :-



$l = 6m$

live load = $20\ kN/m$

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

$f_y = 415\ N/mm^2$

$A_{st} = 5 \times \pi/4 \times 16^2 = 1005.3\ mm^2$

<u>short term (or) Immediate deflection</u> :-

$$\delta = \frac{5wl^4}{384\, E_c\, I_{eff}}$$

$w = L.L + D.L$

$= 20 + (0.3 \times 0.775 \times 25) = 25.812\ kN/m$

L_{eff} = clear span + effective depth

$= 6 + 0.74 = 6.74\ m$

$E_c = 5000\sqrt{f_{ck}} = 5000\sqrt{20} = 0.22 \times 10^5\ N/mm^2$

$$I_{eff} = \frac{I_r}{1.2 - \frac{M_r}{M}\frac{Z}{d}\left(1 - \frac{n_a}{d}\right)\frac{b_w}{b}}$$ [Pg No: 88]

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

$$m = \frac{280}{3\sigma_{cbc}} = \frac{280}{3 \times 7} = 13.33$$

$$\frac{b n_a^2}{2} = m A_{st} (d - n_a)$$

$$\frac{300 \times n_a^2}{2} = 13.33 \times 1005.3 (740 - n_a)$$

$$150 n_a^2 + 13404 n_a - 9918960 = 0$$

$$n_a = 216.32 \text{ mm}$$

$$\therefore I_r = \frac{300 \times 216.32^3}{3} + 13.33 \times 1005.3 (740 - 216.32)^2$$

$$= 46.87 \times 10^8 \text{ mm}^4$$

$$M_r = \frac{f_{cr} \times I_{gross}}{y_t} \quad [\text{Pg No: 16}]$$

$$f_{cr} = \text{Modulus of rupture} = 0.7 \sqrt{f_{ck}} \quad (\text{Pg } 16)$$

$$= 0.7 \sqrt{20} = 3.13 \text{ N/mm}^2$$

$$I_{gross} = \frac{bD^3}{12} = \frac{300 \times 775^3}{12} = 1.16 \times 10^{10} \text{ mm}^4$$

$$y_t = D/2 = 775/2 = 387.5 \text{ mm}$$

$$\therefore M_r = \frac{3.13 \times 1.16 \times 10^{10}}{387.5} = 93.69 \text{ KNm}$$

$$M = \frac{wl^2}{8} = \frac{25.812 \times 6.74^2}{8} = 146.57 \text{ KNm.}$$

$$Z = \text{lever arm} = d - n_a/3 = 740 - \frac{216.32}{3} = 667.92 \text{ mm}$$

$$b_w = b = 300 \text{mm.}$$

$$\therefore I_{eff} = \frac{46.87 \times 10^8}{1.2 - \frac{93.69 \times 10^6}{146.57 \times 10^6} \times \frac{667.92}{740} \left(1 - \frac{216.32}{740}\right) \times \frac{300}{300}}$$

$$= \frac{46.87 \times 10^8}{0.791} = 59.2 \times 10^8 \text{ mm}^4$$

$$\therefore \delta = \frac{5 \times 25.812 \times 6.74^2 \times 1000^4}{384 \times 0.22 \times 10^5 \times 59.2 \times 10^8}$$

$$= 5.32 \text{ mm}$$

Long term deflection:-

(a) Deflection due to shrinkage: (Page 88)

$$a_{cs} = K_3 \psi_{cs} l^2$$

$K_3 = 0.125$ (For simply supported beam)

$$\psi_{cs} = K_4 \frac{\epsilon_{cs}}{D}$$

$$K_4 = 0.72 \frac{p_t - p_c}{\sqrt{p_t}} , \quad 0.25 \leq p_t - p_c < 1.0$$

$$p_t = \frac{100 A_{st}}{bd} = \frac{100 \times 1005.32}{300 \times 740} = 0.45$$

For singly reinforced beam, $p_c = 0$

$$K_4 = 0.72 \times \frac{0.45 - 0}{\sqrt{0.45}} = 0.483$$

$\epsilon_{cs} = 0.0003$ (Pg 16, cl 6.2.4.1)

$$\psi_{cs} = 0.483 \times \frac{0.0003}{775}$$

$$= 1.87 \times 10^{-7}$$

$$a_{cs} = 0.125 \times 1.87 \times 10^{-7} \times 6.74^2 \times 1000^2$$

$$= 1.06 \text{ mm}$$

(b) Deflection due to creep :- (Pg 89)

$$a_{cc\ perm} = a_{icc\ perm} - a_{i\ perm}$$

$$E_{ce} = \frac{E_c}{1+\theta}$$

θ = Creep Coefficient

Assuming the age of loading = 28 days , $\theta = 1.6$

$$E_{ce} = \frac{0.22 \times 10^5}{1+1.6} = 8461.5\ N/mm^2$$

weight due to permenant loads, w_{per} = D.L + 50% L.L

$$= 5.812 + 0.5 \times 20$$

$$= 15.812\ KN/m.$$

Maximum bending moment, $M = \frac{wl^2}{8} = \frac{15.812 \times 6.74^2}{8}$

$$= 89.78\ KNm.$$

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

$$= \frac{46.87 \times 10^8}{1.2 - \frac{93.69}{89.78} \times \frac{667.92}{740}\left(1 - \frac{216.32}{740}\right) \times 1}$$

$$= \frac{46.87 \times 10^8}{0.533} = 87.86 \times 10^8\ mm^4$$

$$a_{icc} = \frac{5wl^4}{384\ E_{ce}\ I_{eff}}$$

$$= \frac{5 \times 15.812 \times 6740^4}{384 \times 8461.5 \times 87.86 \times 10^8}$$

$$= 5.71\ mm.$$

Initial deflection, $a_i = \dfrac{5 w l^4}{384 E_c I_{eff}}$

$$= \frac{5 \times 15.812 \times 6740^4}{384 \times 0.22 \times 10^5 \times 87.86 \times 10^8}$$

$$= 2.17 \text{ mm}$$

$$a_{cc} = a_{icc} - a_i$$

$$= 5.71 - 2.17$$

$$= 3.54 \text{ mm}$$

$\therefore$ Total longterm deflection $= 3.54 + 1.06 = 4.6$ mm

$\therefore$ Total deflection = short term deflection + Long term deflection.

$$= 5.32 + 4.6 = 9.92 \text{ mm}$$

Permissible deflection as per IS Code $= \dfrac{L}{250}$

$$= \frac{6740}{250} = 26.96 \text{ mm}$$

Permissible deflection after erection of partition walls $= \dfrac{L}{350}$

$$= \frac{6740}{350} = 19.25 \text{ mm}$$

$\therefore$ The Section is <u>safe</u> in deflection

Problem on both cracking & deflection

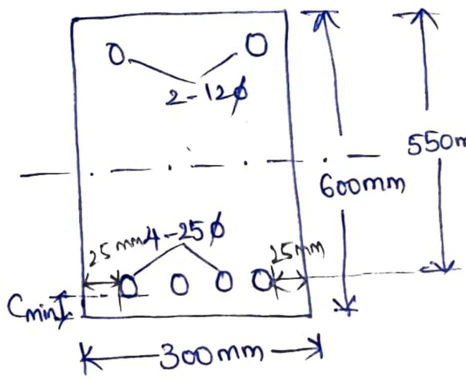
2) A Simply supported beam spanning over 8m is of rectangular section with a width of 300 mm and overall depth of 600mm. The beam is reinforced with 4 bars of 25 mm diameter on the tension side at an effective depth of 550mm. Two nominal hanger bars of 12mm diameter are provided on the compression side. The beam is subjected to a service moment of 140 KNm at the centre of the span section. Use M_{20} and Fe_{415} materials. check the beam for the serviceability limit states of deflection and cracking using the following methods.

(i) Deflection Controls (Emperical method)

(ii) Deflection Control (theoritical method)

(iii) Maximum width of cracks (Theoritical method).

Sol:-



(iii) Maximum width of cracks :-

$$W_{cr} = \frac{3\, a_{cr}\, \epsilon_m}{\left[1 + 2\frac{(a_{cr} - C_{min})}{h-x}\right]}$$

$C_{min} = 50 - 12.5 = 37.5$ mm

$$a_{cr} = \left[(0.5\, s)^2 + C_{min}^2\right]^{0.5}$$

$$s = \text{spacing} = \frac{300 - (4\times25 + 2\times25)}{3} = 50\text{mm}$$

$$a_{cr} = \left[(0.5\times50)^2 + 37.5^2\right]^{0.5}$$

$= 45.06$mm.

Crack width will be maximum at the soffit of the beam,

$$a' = h = 600mm$$

$$\epsilon_1 = \frac{f_s}{E_s}\frac{(h-x)}{(d-x)}$$

$$f_s = m \times \frac{M \times y}{I_x}$$

$$m = \frac{280}{3\sigma_{cbc}} = \frac{280}{3 \times 7} = 13.33$$

$$M = 140 \text{ KNm}$$

$$(m-1)A_{sc}(n_a-d_c) + \frac{b n_a^2}{2} = m \times A_{st} \times (d - n_a)$$

$$12.33 \times 226.19(n_a-50) + \frac{300 \times n_a^2}{2} = 13.33 \times 4 \times \pi/4 \times 25^2 \times (550 - n_a)$$

$$\Rightarrow n_a = 229.37 mm$$

$$y = 550 - 229.37 = 320.63 mm$$

$$I_x = \frac{b n_a^3}{3} + m A_{st} (d - n_a)^2 + (m-1) A_{sc} (n_a - d_c)^2$$

$$= \frac{300 \times 234^3}{3} + 13.33 \times 4 \times \pi/4 \times 25^2 (550 - 229.37)^2$$

$$= 39.87 \times 10^8 \text{ mm}^4$$

$$\therefore f_s = 13.33 \times \frac{140 \times 10^6 \times 316}{38.94 \times 10^8} = 150.97 \text{ N/mm}^2$$

$$\therefore \epsilon_1 = \frac{151.14}{2 \times 10^5}\frac{(600 - 229.37)}{(550 - 229.37)} = 8.68 \times 10^{-4}$$

$$\therefore \epsilon_m = \epsilon_1 - \frac{b(h-x)(a-x)}{3 E_s A_{st}(d-x)}$$

$$= 8.63 \times 10^{-4} - \frac{300(600-234)(600-229.3)}{3 \times 2 \times 10^5 \times 4 \times \pi/4 \times 25^2 (550 - 229.3)} = 7.50 \times 10^{-4}$$

$$W_{cr} = \frac{3\, a_{cr}\, \epsilon_m}{\left[1 + \frac{2(a_{cr} - C_{min})}{h - x}\right]} = \frac{3 \times 45.06 \times 7.60 \times 10^{-4}}{\left[1 + \frac{2(45.06 - 37.5)}{600 - 229.37}\right]}$$

$$= \frac{0.104}{1.04} = 0.1\ mm$$

Maximum width of crack should not be greater than $0.004 \times C_{min}$

$= 0.15\ mm$

$= 0.004 \times 37.5 = 0.15\ mm > 0.1\ mm\ (W_{cr})$

Hence the section is safe.

∴ The serviceability limit state of cracking is satisfied.

(i) Deflection Control (empirical method):-

$$(L/d)_{max} = (L/d)_{basic} \times K_t \times K_c \times K_f$$

$(L/d)_{basic} = 20$ for simply supported beam

$K_f = 1$ (From Fig 6, Pg 39)

$K_t = 0.95$ (From Fig 4, Pg 38)

$K_c = 1.05$ (From fig 5, Pg 39)

$$(L/d)_{max} = 20 \times 0.95 \times 1.05 \times 1 = 19.95$$

$$(L/d)_{prov} = \frac{8000}{550} = 14.54$$

$$(L/d)_{max} > (L/d)_{prov}$$

Hence the section is safe in deflection.

(ii) Deflection Control (Theoritical method) :-

(a) short term deflection :-

$$\delta = \frac{5 w l^4}{384 E_c I_{eff}}$$

$$M = \frac{w l^2}{8} \Rightarrow 140 = \frac{w \times 8.55^2}{8}$$ ($l_{eff} = 8 + 0.55 = 8.55m$)

$$\Rightarrow w = 15.32 \ kN/m$$

$$E_c = 5000\sqrt{f_{ck}} = 5000\sqrt{20} = 0.22 \times 10^5 \ N/mm^2$$

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

$I_r = 39.87 \times 10^8 \ mm^4$, $n_a = 229.37 mm$

$$M_r = \frac{f_{cr} I_g}{y_t}$$

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

$$M_r = \frac{3.13 \times \frac{300 \times 600^3}{12}}{300} = 56.34 \ kN m$$

$$z = d - n_a/3 = 550 - \frac{229.37}{3} = 473.5 mm$$

$$I_{eff} = \frac{39.87 \times 10^8}{1.2 - \frac{56.34}{140} \times \frac{472}{550}\left(1 - \frac{229.37}{550}\right) \times 1}$$

$$= \cancel{46.56}\ 39.94 \times 10^8 \ mm^4$$

$$\delta = \frac{5 \times 15.32 \times 8.55^4 \times 1000^4}{384 \times 0.22 \times 10^5 \times \cancel{46.56}\ 39.94 \times 10^8}$$

$$= \cancel{10.4}\ 12.13 \ mm$$

(b) Long term deflection :-

(i) Deflection due to shrinkage :-

$$a_{cs} = K_3 \psi_{cs} l^2$$

$K_3 = 0.125$ For simply supported beam

$$\psi_{cs} = K_4 \times \frac{\epsilon_{cs}}{D}$$

$$K_4 = 0.65 \times \frac{p_t - p_c}{\sqrt{p_t}} \quad , \quad p_t - p_c \geq 1.0$$

$$p_t = \frac{100 \times \pi/4 \times 25^2 \times 4}{300 \times 550} = 1.189\%$$

$$p_c = \frac{100 \times \pi/4 \times 12^2 \times 2}{300 \times 550} = 0.137\%$$

$$K_4 = 0.65 \times \frac{1.189 - 0.137}{\sqrt{1.189}} = 0.627$$

$$\epsilon_{cs} = 0.0003$$

$$\psi_{cs} = 0.627 \times \frac{0.0003}{600} = 3.135 \times 10^{-7}$$

$$a_{cs} = 0.125 \times 3.135 \times 10^{-7} \times 8.55^2 \times 1000^2$$

$$= 2.86 \text{ mm}$$

(ii) Deflection due to creep :-

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

$$E_{ce} = \frac{E_c}{1+\theta}$$

θ = creep Coefficient = 1.6

$$E_{ce} = \frac{0.22 \times 10^5}{2.6} = 8461.53 \text{ N/mm}^2$$

Dead load $= 0.6 \times 0.3 \times 25 = 4.5$ kN/m

$w = 15.32$ kN/m

Live load $= 15.32 - 4.5 = 10.82$ kN/m

weight due to permenant loads, $w_{perm} = D.L + 50\%\ L.L$

$$= 4.5 + \frac{10.82}{2} = 9.91 \text{ kN/m}$$

$$M_{perm} = \frac{9.91 \times 8.55^2}{8} = 90.55 \text{ kNm}$$

$$I_{eff} = \frac{\overset{39.87}{\cancel{38.97}} \times 10^8}{1.2 - \frac{56.34}{90.55} \times \frac{473.54}{550}\left(1 - \frac{229.37}{550}\right) \times 1}$$

$$= 44.92 \times 10^8 \text{ mm}^4$$

$$a_{icc} = \frac{5 w l^4}{384\, E_{ce}\, I_{eff}} = \frac{5 \times 9.91 \times 8550^4}{384 \times 8461.53 \times 44.92 \times 10^8} = 18.04 \text{ mm}$$

Initial deflection, $a_i = \dfrac{5 \times 9.91 \times 8550^4}{384 \times 0.22 \times 10^5 \times 44.92 \times 10^8} = \overset{6.97}{\cancel{7.02}}$ mm

$a_{cc} = a_{icc} - a_i = 18.04 - 6.97 = 11.07$ mm

Total longterm deflection $= 11.07 + 2.86 = 14.03$ mm

$\therefore$ Total deflection $= 14.03 + \overset{12.13}{\cancel{10.4}} = \overset{26.16}{\cancel{24.49}}$ mm

Permissible deflection as per code $= l/250 = \dfrac{8550}{250} = 34.2$ mm

permissible deflection after erection of partition walls $= \dfrac{l}{350}$

$$= \frac{8550}{350} = 24.43 \text{ mm}$$

$\therefore$ The <u>section</u> is safe.