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DESIGN PROCEDURE FOR DOUBLY REINFORCED BEAM

DESIGNING DOUBLY REINFORCED BEAM FOR SHEAR STRESS

Step 1: Determining nominal shear stress

The nominal shear stress in beams of uniform depth shall be obtained by the following equation.

$\tau_v = V_u / b_d$

Where,

V_u = shear force due to design load

b = breadth of the member, which for flanged section shall be is taken as the breadth of the web, b_w, and

d = effective depth

FOR SHEAR STRESS

For solid slabs, the design shear strength for concrete shall be $\tau_c k$, where k has values given below:

NOTE: **This provision shall not apply to flat slabs**

Under no circumstances even with the shear reinforcement, shall the nominal shear stress in beams $\tau_{v\,exceed}$ τ_{cmax} given in table 20 of IS 456: 2000

Overall depth of slab, mm	**300 or more**	**275**	**250**	**225**	**200**	**175**	**150 or less**
k	1.00	1.05	1.10	1.15	1.20	1.25	1.30

minimum shear reinforcement

When τ_v is less than τ_c given in table 19 of IS 456:2000, minimum shear reinforcement shall be provided

design of shear reinforcement

When τ_v exceeds τ_v given in table 19, shear reinforcement shall be provided in any of the following forms:

- Vertical stirrups,
- Bent-up bars along with stirrups, &
- Inclined stirrups

Shear reinforcement shall be provide to carry a shear equal to

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

STRENGTH OF SHEAR REINFORCEMENT

- The strength of shear reinforcement V_{us} shall be calculated as below:

- For vertical stirrups:
- $V_{us} = 0.87 f_y A_{sv} d / s_v$

- For inclined stirrups or a series of bars bent up at different cross sections:
- $V_{us} = (0.87 f_y A_{sv} d / s_v)(\sin \alpha + \cos \alpha)$

- For single bar or single group of parallel bars, all bent up at the cross-section:
- $V_{us} = 0.87 f_y A_{sv} \sin \alpha$
- Where,
- A_{sv} = s_v = τ_v =
- τ_c = b = f_y =
- α = d =

FOR FLEXTURE

1. Design a rectangular reinforced concrete beam to carry a factored BM of 200KN-m, factored shear force of 120kN & factored torsion moment of 75 KN-m Assume M-20 concrete & Fe 415 steel

Sol: M_u = 20KN-m, T_u=75KN-m, V_u=120K.N.

Step -1= Design of BM & Torsion

Assume the ratio $\frac{D}{b}$=2

$$M_T = \frac{T_u(1+\frac{D}{b})}{l_o 7} = \frac{75(1+2)}{1.7} = 132.35KN - m.$$

No compression reinforcement design is necessary

$M_T < M_u$

$M_{e1} = M_T + M_u = 200 + 132.35 = 332.35$KN-m.

$d_{bal} = \sqrt{\frac{M_{e1}}{\theta_{lim} b}} = \sqrt{\frac{332.35 \times 10^6}{2.76 \times 300}} = 633.5$mm.

Assume b=300mm, θ_{lim}=2.76

Assuming overall depth as 700mm & width as 350mm & effective cover =50mm.

D Provided =700-50

=650mm

Area of steel required for under reinforced section,

$P_t = \frac{50 f_{ck}}{fy}\left[1 - \sqrt{1 \frac{4.6 M_{e1}}{f ck b d^2}}\right]$

$= \frac{50 \times 20}{415}\left[1 - \sqrt{1 \frac{4.6 \times 332.35 \times 10^6}{20 \times 350 \times 650^2}}\right]$

$= 0.73 < 0.96$

(p_t lim)

$\therefore A_{st} = \frac{0.73 \times 350 \times 650}{100} = 1660.75 mm^2$

Assume 25mm dia bars = $\frac{1660.75}{4.91}$=3.38≈4

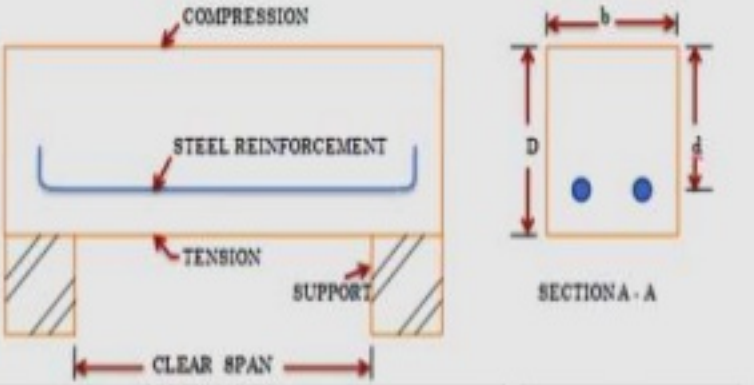
Provide 4 bars -#25 dia

(A_{st}) provided = 1963mm^2

DESIGN OF R.C.C. BEAM

FOR SHEAR AND FLEXTURE.

- BEAM
- TYPES OF BEAMS
- SINGLY REINFORCED BEAM (FLEXURE AND SHEAR)
- DOUBLY REINFORCED BEAM (FLEXURE AND SHEAR)



AAKANSHA 1216512101
ANKITHA 1216512103
ARUNDATHI 1216512104
ASHOK SAHOO 1216512105

BEAM

- Beam is a horizontal member of structure carrying transverse loads.
- Beam is rectangular in cross-section.
- Beams carry the floor slab or roof slab.
- Beams transfers all the loads including its self weight to the columns or walls.

R.C.C. BEAM

- RCC beam is subjected to bending moments and shear.
- Due to the vertical external load, bending compresses the top fibers of the beam and elongates the bottom fibers.
- The strength of the RCC beam depends upon the composite action of concrete and steel.

WORKING STRESS METHOD

- The Stresses in an element is obtained from the working loads and compared with permissible stresses.
- The method follows linear stress-strain behavior of both the materials.
- Modular ratio can be used to determine allowable stresses.
- Material capabilities are under estimated to large extent. Factor of safety are used in working stress method.
- The member is considered as working stress.
- Ultimate load carrying capacity cannot be predicted accurately.
- The main drawback of this method is that it results in an uneconomical section

LIMIT STATE METHOD

- The stresses are obtained from design loads and compared with design strength.
- In this method, it follows linear strain relationship but not linear stress relationship (one of the major difference between the two methods of design).
- The ultimate stresses of materials itself are used as allowable stresses.
- The material capabilities are not under estimated as much as they are in working stress method. Partial safety factors are used in limit state method.

LOAD ACTING ON A STRUCTURE:

DEAD LOAD:

❖ DEAD Load is self-weight of the various components in the building.

LIVE LOAD:

❖ LIVE Load is the external superimposed load on a structure.

- Uniformly distributed load.
- Uniformly varying load.
- Concentrated load.
- Arbitrary load.

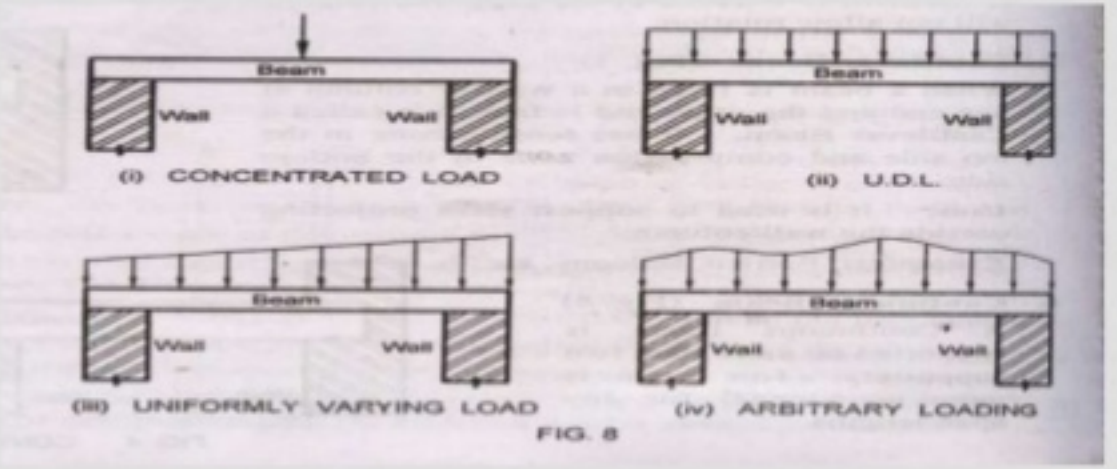
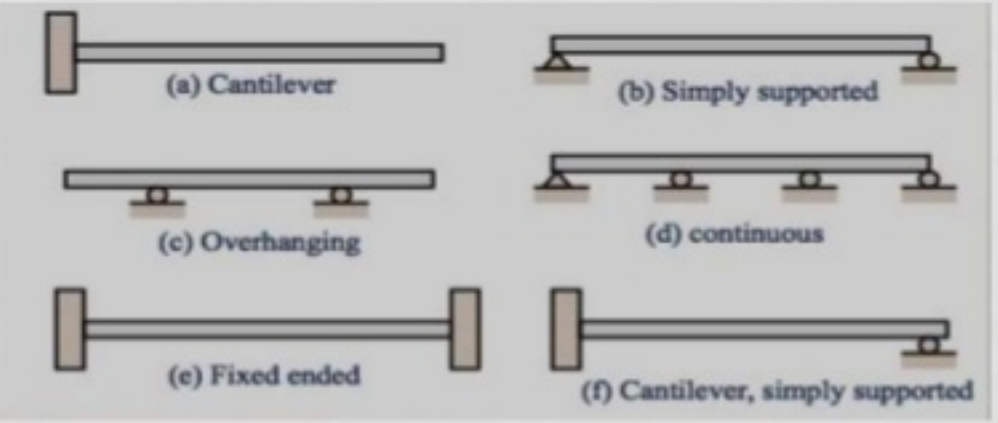


FIG. 8

TYPES OF BEAMS:

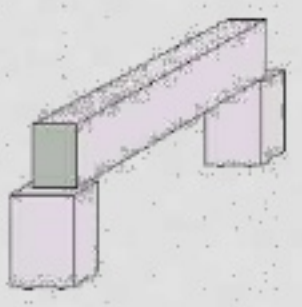
- Simply Supported Beam.
- Fixed Beam.
- Cantilever Beam.
- Continuous Beam.
- Overhanging Beam.
- Cantilever, Simply supported beam



SIMPLY SUPPORTED BEAM:

- It is a beam that is freely supported at two ends on walls or columns.

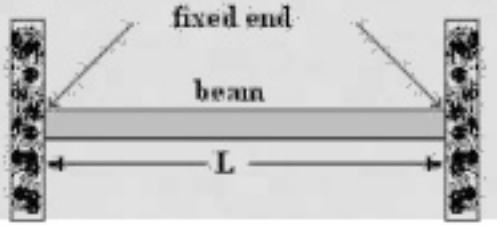
- In actual practice no beam rests freely on the supports (columns or walls)



FIXED BEAM:

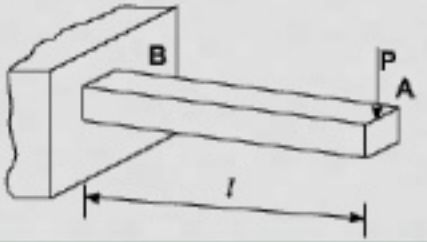
- In this beam both ends of the beam are rigidly fixed into the supports.

- Main reinforcement bars and stirrups are also provided.



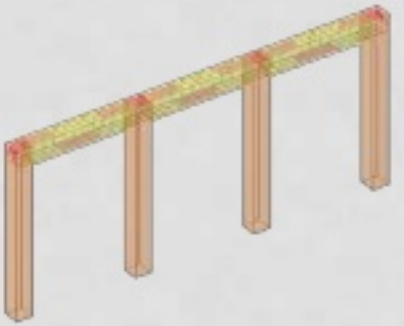
CANTILEVER BEAM:

- One end of the beam is fixed to wall or column and the other end is free.
- It has tension on top and compression on bottom.



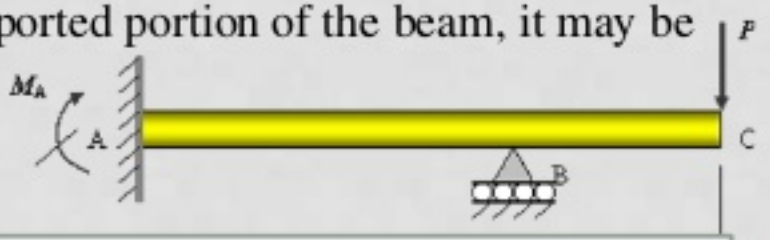
CONTINIOUS BEAM:

- A **continuous beam** is a statically indeterminate multi span **beam** on hinged support.
- The end spans may be cantilever, may be freely supported or fixed supported. At least one of the supports of a continuous **beam** must be able to develop a reaction along the **beam** axis.



OVERHANGING BEAM:

- In overhanging beam its end extends beyond column or wall support.
- Overhanging of the beam is the unsupported portion of the beam, it may be on side or both the sides.



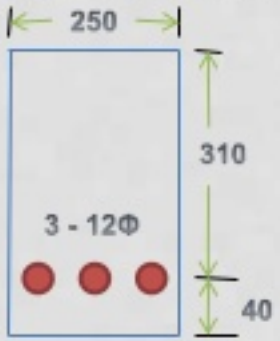
TYPES OF RCC BEAMS:

- SINGLY REINFORCED BEAM:
  - •Singly reinforcement beam have steel provided only one side tension an another side compression. tension takes steel load or tensile load and compression takes concrete or compressive load.
- DOUBLY REINFORCED BEAM:
  - •Doubly reinforced sections contain reinforcement both at the tension and at the compression face, usually at the support section only.

SINGLY REINFORCED BEAM

- Determine the moment of resistance for the section shown in figure.

(i) f_{ck} = 20 N/mm , f_y = 415 N/mm



Solution:

(i) f_{ck} = 20 N/mm , f_y = 415 N/mm

breadth (b) = 250 mm

effective depth (d) = 310 mm

effective cover = 40 mm

Force of compression = 0.36 f_{ck} b x

= 0.36 X 20 X 250x

= 1800x N

Area of tension steel A_t = 3 X 113 mm

Force of Tension = 0.87 f_y A_t

= 0.87 X 415 X 3 X 113

= 122400 N

- Force of Tension = Force of compression
- 122400 = 1800x
- x = 68 mm
- x_m = 0.48d
- = 0.48 X 310
- = 148.8 mm
- 148.8 mm > 68 mm
- Therefore,
- Depth of neutral axis = 68 mm

f_y	x_m
415	0.48d
500	0.46d

SINGLY REINFORCED BEAM

Lever arm z = d – 0.42x
= 310 – 0.42 X 68
= 281 mm

As x < xm (It is under reinforced)

Since this is an under reinforced section,
moment of resistance is governed by steel.

Moment of resistance w.r.t steel = tensile force X z
Mu = 0.87*fy* At z
= 0.87 X 415 X 3 X 113 X 281
Mu = 34.40kNm

DESIGN FOR SHEAR

- Question : Design a rectangular beam to resist a bending moment equal to 45 kNm using (i) M15 mix and mild steel

- *Solution :*
- The beam will be designed so that under the applied moment both materials reach their maximum stresses.

- Assume ratio of overall depth to breadth of the beam equal to 2.

  Breadth of the beam = b
  Overall depth of beam = D
  therefore , D/b = 2

For a balanced design,

Factored BM = moment of resistance with respect to concrete
= moment of resistance with respect to steel
= load factor X B.M
= 1.5 X 45
= 67.5 kNm

DESIGN FOR SHEAR(CONT.)

For balanced section,

Moment of resistance M_u = $0.36 f_{ck} b x_m(d - 0.42 x_m)$

Grade for mild steel is Fe250

For Fe250 steel,

$x_m = 0.53d$

$M_u = 0.36 fck\ b\ (0.53\ d)\ (1 - 0.42\ X\ 0.53)\ d$

$= 2.22bd$

Since D/b =2 or, d/b = 2 or, b=d/2

$Mu = 1.11\ d$

$Mu = 67.5\ X\ 10$ Nmm

d=394 mm and b= 200mm

DESIGN FOR SHEAR(CONT.)

- Adopt D = 450 mm
- b = 250 mm
- d = 415mm

❑ Area of tensile steel At = $\dfrac{\text{Factored BM}}{0.87\, f_y\, (d - 0.42\, x_m)}$

❑ = $\dfrac{67.5 \times 10}{0.87 \times 250\, (1 - 0.42 \times 0.53) 415}$

= 962 mm

= 9.62 cm

❑ Minimum area of steel A_o= 0.85

DESIGN FOR SHEAR(CONT.)

$$= \frac{0.85 \times 250 \times 415}{250}$$

$$= 353 \text{ mm}$$

$$353 \text{ mm} < 962 \text{ mm}$$

In beams the diameter of main reinforced bars is usually selected between 12 mm and 25 mm.

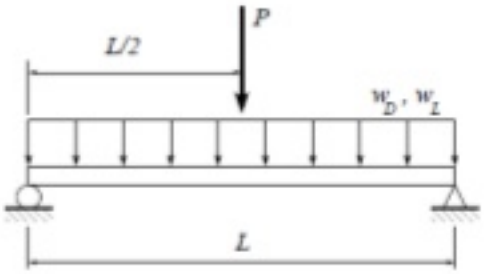
Provide 2-20mm and 1-22mm bars giving total area

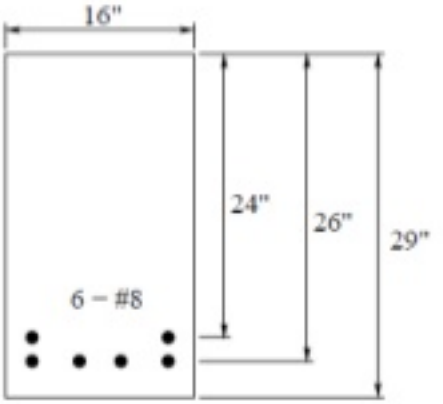
$$= 6.28 + 3.80$$

$$= 10.08 \text{ cm} > 9.62 \text{ cm}$$

ANALYSIS FOR SRB FLEXURE

Your office is designing a simply supported beam subject to the loading shown below. The point load is a live load with $P = 15$ kip, the distributed loads are $w_D = 0.7$ kip/ft and $w_L = 1.2$ kip/ft. The span length is $L = 30$ ft. Your co-worker has given you the beam design shown below to check. Material strengths are: $f'_c = 4$ kip/in^2, $f_y = 60$ kip/in^2. Is the beam adequate?





1. **Factor Loads, Compute M_u**

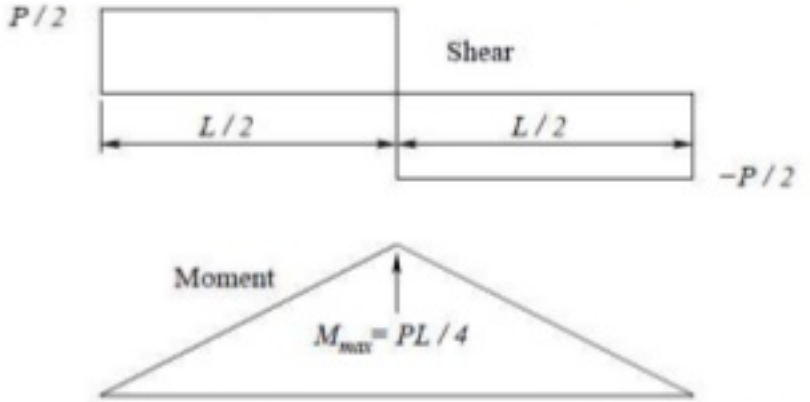
We will compute the maximum factored load, M_u, by considering the distributed and point loads separately. The service (unfactored) distributed dead load, w_D, is less than eight times the distributed live load, w_L, and there are no other load types, so equation (9-2) of {ACI 9.2.1} controls when calculating the factored distributed load, w_u:

$$w_u = 1.2w_D + 1.6w_L = 1.2(0.7\text{ kip/ft}) + 1.6(1.2\text{ kip/ft}) = 2.8\text{ kip/ft} \quad (1)$$

For a constant distributed load, the maximum moment occurs at midspan. We are interested in the worst case, so this is the moment that will be used as the factored load, $M_{u,dist}$:

$$M_{u,dist} = \frac{w_u L^2}{8} = \frac{2.8\text{ kip/ft}(30\text{ ft})^2}{8} = 311\text{ kip-ft} \quad (2)$$

Considering just the live point load, the shear and moment diagrams are:



The maximum moment occurs at midspan just as with the distributed loading and is equal to $PL/4$. To get the maximum factored load, use $P_u = 1.6P$ in place of P:

$$M_{u,point} = \frac{P_u L}{4} = \frac{(1.6P)L}{4} = \frac{1.6(15\text{ kip})30\text{ ft}}{4} = 180\text{ kip-ft} \quad (3)$$

The total factored moment is the sum of the factored moments from the distributed loading and the point loading in Equation 2 and Equation 3, respectively.

$$M_u = M_{u,dist} + M_{u,point} = 311 \text{ kip-ft} + 180 \text{ kip-ft} = 491 \text{ kip-ft} \tag{4}$$

2. **Check minimum steel.** Use a weighted average to determine d:

$$d = \frac{4(26 \text{ in}) + 2(24 \text{ in})}{6} = 25.3 \text{ in} \tag{5}$$

Now calculate A_s and ensure it is more than the minimum.

$$A_s = 6(0.79 \text{ in}^2) = 4.74 \text{ in}^2 \tag{6}$$

$$A_{s,\min} = \frac{3\sqrt{f'_c}}{f_y} b_w d = \frac{3\sqrt{4,000} \text{ lb/in}^2}{60,000 \text{ lb/in}^2} 16 \text{ in}(25.3 \text{ in}) = 1.28 \text{ in}^2 < A_s \tag{7}$$

$$\frac{200 b_w d}{f_y} = \frac{200 \text{ lb/in}^2 (16 \text{ in})(25.3 \text{ in})}{60,000 \text{ lb/in}^2} = 1.35 \text{ in}^2 < A_s \tag{8}$$

The steel area, A_s, is greater than both required minimums from the code.

3. **Compute T, C, a, c.** Assuming the steel has yielded:

$$T = f_y A_s = 60 \text{ kip/in}^2 (4.74 \text{ in}^2) = 284 \text{ kip} \tag{9}$$

By equilibrium, $C = T$. Calculating the compressive force, C, as the equivalent force from the concrete compressive stress:

$$C = 0.85 f'_c a b \tag{10}$$

Solving for a:

$$a = \frac{C}{0.85 f'_c b} = \frac{T}{0.85 f'_c b} = \frac{284 \text{ kip}}{0.85(4 \text{ kip/in}^2)(16 \text{ in})} = 5.2 \text{ in} \tag{11}$$

By equilibrium, $C = T$. Calculating the compressive force, C, as the equivalent force from the concrete compressive stress:

$$C = 0.85 f'_c ab \tag{10}$$

Solving for a:

$$a = \frac{C}{0.85 f'_c b} = \frac{T}{0.85 f'_c b} = \frac{284 \text{ kip}}{0.85(4 \text{ kip/in}^2)(16 \text{ in})} = 5.2 \text{ in} \tag{11}$$

For 4,000 psi concrete, $\beta_1 = 0.85$:

$$c = \frac{a}{\beta_1} = \frac{5.2 \text{ in}}{0.85} = 6.2 \text{ in} \tag{12}$$

4. **Check minimum steel strain.**

$$\frac{c}{d_t} = \frac{6.2 \text{ in}}{26 \text{ in}} = 0.237 \leq 0.375 \tag{13}$$

Section is tension-controlled. Thus, the steel has yielded as assumed in the previous step and the section meets minimum steel strain requirements of the code.

5. **Compute M_n, check $\phi M_n \geq M_u$.** For a tension-controlled section: $\phi = 0.9$.

$$M_n = T\left(d - \frac{a}{2}\right) = 284 \text{ kip}\left(25.3 \text{ in} - \frac{5.2 \text{ in}}{2}\right)\left(\frac{1 \text{ ft}}{12 \text{ in}}\right) = 538 \text{ kip-ft} \tag{14}$$

$$\phi M_n = 0.9(538 \text{ kip-ft}) = 484 \text{ kip-ft} < M_u = 491 \text{ kip-ft} \tag{15}$$

The beam is **not** adequate—it should be redesigned for greater strength.

DESIGN FOR FLEXTURE

Design a rectangular reinforced concrete beam for loads given below. The simply-supported beam has a span $\ell = 18$ ft and excessive deflections will cause damage. The superimposed dead load (SDL) is 1.15 kip/ft with other given quantities below.

Given: $f'_c = 4.5$ kip/in^2 $\quad w_L = 0.8$ kip/ft
$\gamma_c = 150$ lb/ft^3 $\quad f_y = 60$ kip/in^2

FLEXURAL DESIGN

(A) Choose the beam depth, h. Because excessive deflections could cause damage, we will not use Table 9.5(a) from the code. Instead, use $h = \ell/12 = 18$ ft$/12$ (12 in/foot) $= 18$ in.

(B) Assume one row of steel: $d = h - 2.5$ in $= 18$ in $- 2.5$ in $= 15.5$ in.

(C) Assuming $r = b/d = 0.5$, $b = 0.5d = 0.5(15.5 \text{ in}) = 7.75$ in, use $b = 8$ in.

(D) Compute M_u and M_n. With our estimated beam height and width, we can calculate the self-weight, SW, and determine the total dead load, w_D.

$$\begin{aligned} SW &= bh\gamma_c \\ &= (8\text{ in})(18\text{ in})(150\text{ lb/ft}^3)(1\text{ foot}^2/144\text{ in}^2)(1\text{ kip}/1000\text{ lb}) \\ &= 0.15\text{ kip/ft} \\ w_D &= SDL + SW = (1.15\text{ kip/ft} + 0.15\text{ kip/ft}) \\ &= 1.3\text{ kip/ft} \end{aligned}$$

Because only dead and live loads are given and the dead load is less than eight times the live load, only one factored load case need be checked:

$$w_u = 1.2w_D + 1.6w_L = 1.2(1.3\text{ kip/ft}) + 1.6(0.8\text{ kip/ft}) = 2.84\text{ kip/ft} \tag{1}$$

Which leads to a factored moment, M_u, for a simply-supported beam of:

$$M_u = \frac{w_u \ell^2}{8} = \frac{2.84\text{ kip/ft}(18\text{ ft})^2}{8} = 115\ \text{ kip-ft} \tag{2}$$

Assuming that $\phi = 0.9$, the required nominal strength, M_n, can be found.

$$M_n = \frac{M_u}{\phi} = \frac{115\ \text{ kip-ft}}{0.90} = 128\ \text{ kip-ft} \tag{3}$$

(E) Compute $R = \frac{M_n}{bd^2}$ and find ρ.

$$R = \frac{M_n}{bd^2} = \frac{128\ \text{ kip-ft}}{(8\text{ in})(15.5\text{ in})^2}(12\text{ in/foot})(1000\text{ lb}/1\text{ kip}) = 798\text{ psi} \tag{4}$$

Reading from the chart gives $\rho \approx 0.0152$.

(F) Compute $A_s = \rho bd$.

$$A_s = \rho bd = 0.0152(8\text{ in})(15.5\text{ in}) = 1.88\text{ in}^2 \tag{5}$$

(G) Choose bars, make sure they fit. Based on the required $A_s = 1.88\text{ in}^2$, the following options are available:

Bar Size	A_b (in^2)	# Bars	A_s (in^2)
6	0.44	5	2.20
7	0.60	4	2.40
8	0.79	3	2.37
9	1.00	2	2.00
10	1.27	2	2.54
11	1.56	2	3.12

Based on these options, choose 2 #9s with a total area of $A_s = 2.00\text{ in}^2$ since it is the smallest area greater than the initial estimate and thus most efficient.

To get the minimum width, assume 1.5 inches clear cover and a #3 stirrup. The minimum beam width, b_{min}, will be the sum of the cover requirements, the stirrup width (on each side of the beam), the flexural reinforcement (2 #9s), and the clear spacing between flexural reinforcing. A #9 bar has $d_b = 1.128$ in so that the minimum clear spacing must be 1.128 in. Adding it all up:

$$b_{\text{min}} = 2(1.5\text{ in}) + 2(0.375\text{ in}) + 3(1.128\text{ in}) = 7.13\text{ in} < b = 8\text{ in} \tag{6}$$

The minimum width, b_{min}, is less than our actual width so the bars will fit.

The minimum width, $b_{\min}$, is less than our actual width so the bars will fit.

(H) Analyze the section, check $\phi M_n \geq M_u$.

(1) Factor loads and compute required M_u. This was computed above: $M_u = 115$ kip-ft.

(2) Check minimum steel. The depth, d, to the centroid of the steel can now be computed exactly.

$$d = 18 \text{ in} - 1.5 \text{ in} - 0.375 \text{ in} - 1.128 \text{ in}/2 = 15.56 \text{ in} \tag{7}$$

The minimum steel area is then:

$$A_{s,\min} = \frac{3\sqrt{f'_c}}{f_y} b_w d = \frac{3\sqrt{4,500} \text{ lb/in}^2}{60,000 \text{ lb/in}^2} 8 \text{ in}(15.56 \text{ in}) \tag{8}$$

$$= 0.418 \text{ in}^2 < A_s = 2.00 \text{ in}^2 \tag{9}$$

$$A_{s,\min} = \frac{200 \text{ lb/in}^2}{f_y} b_w d = \frac{200 \text{ lb/in}^2}{60,000 \text{ lb/in}^2} 8 \text{ in}(15.56 \text{ in}) \tag{10}$$

$$= 0.415 \text{ in}^2 < A_s = 2.00 \text{ in}^2 \tag{11}$$

Minimum steel area requirement is met.

(3) Compute T, C, a, and c. Assuming the tension steel has yielded:

$$T = A_s f_y = 2.00 \text{ in}^2 (60 \text{ kip/in}^2) = 120 \text{ kip} \tag{12}$$

By equilibrium:

$$C = T = 120 \text{ kip} \tag{13}$$

We can now solve for a and c.

$$a = \frac{C}{0.85 f'_c b} = \frac{120 \text{ kip}}{0.85(4.5 \text{ kip/in}^2)(8 \text{ in})} = 3.92 \text{ in} \quad (14)$$

$$c = a/\beta_1 = 3.92 \text{ in}/0.825 = 4.75 \text{ in} \quad (15)$$

(4) Check minimum strain. Because we only have one row, $d = d_t$.

$$\frac{c}{d_t} = \frac{c}{d} = \frac{4.75 \text{ in}}{15.56 \text{ in}} = 0.305 < 0.375 \quad (16)$$

The ratio $\frac{c}{d_t}$ is less than 0.375, thus the section is tension-controlled. A tension-controlled section has $\varepsilon_t > 0.005$, so it follows that $\varepsilon_t > 0.004$ as required. Thus, there is significant yielding of the tension steel, which verifies our assumption from the previous step that the tension steel had yielded. The minimum strain requirement is met.

(5) Compute M_n, check $\phi M_n \geq M_u$. The section is tension-controlled (from the previous step) so that $\phi = 0.9$.

$$\phi M_n = \phi T(d - a/2) = 0.9(120 \text{ kip})(15.56 \text{ in} - 3.92 \text{ in}/2)(1 \text{ foot}/12 \text{ in}) \quad (17)$$

$$= 122.4 \text{ kip·ft} > M_u = 115 \text{ kip·ft} \quad (18)$$

The beam is strong enough ($\phi M_n > M_u$) and all other code checks were satisfactory. Thus, the beam is adequate for the loads.