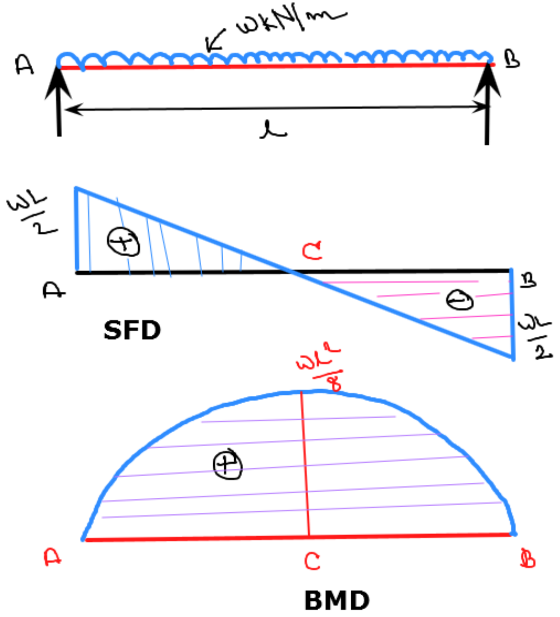
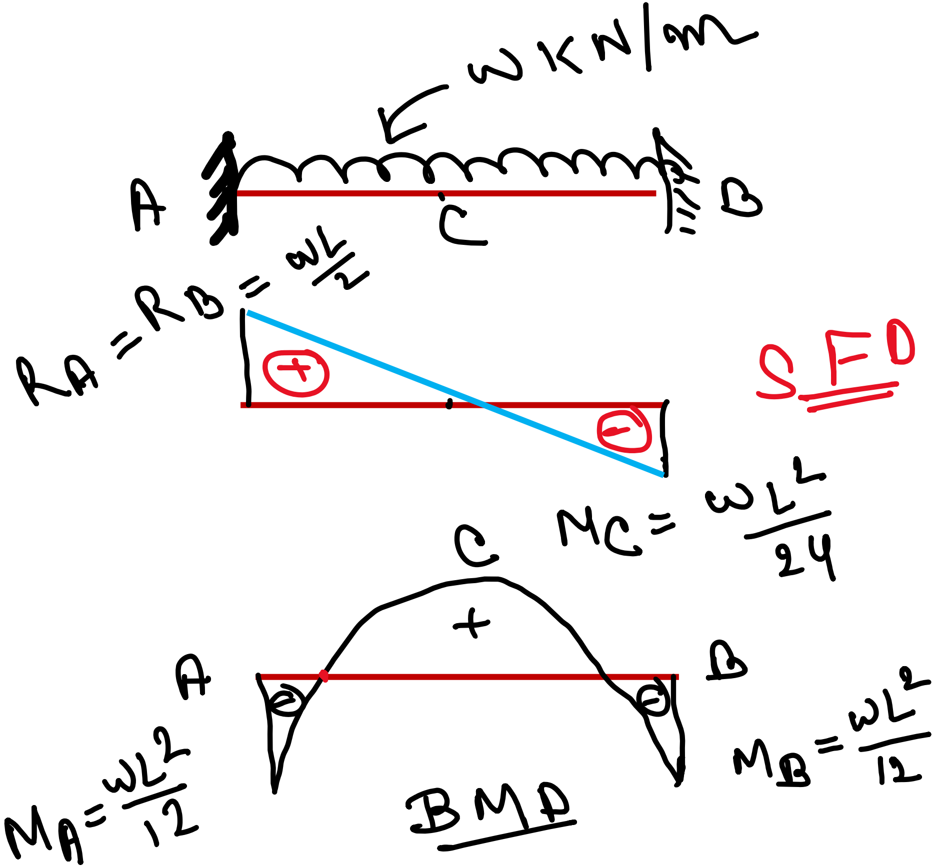
Design of Reinforced Concrete Structures

INTRODUCTION TO DESIGN OF SINGLY REINFORCED BEAM AS PER IS 456

DEBASIS PANDA

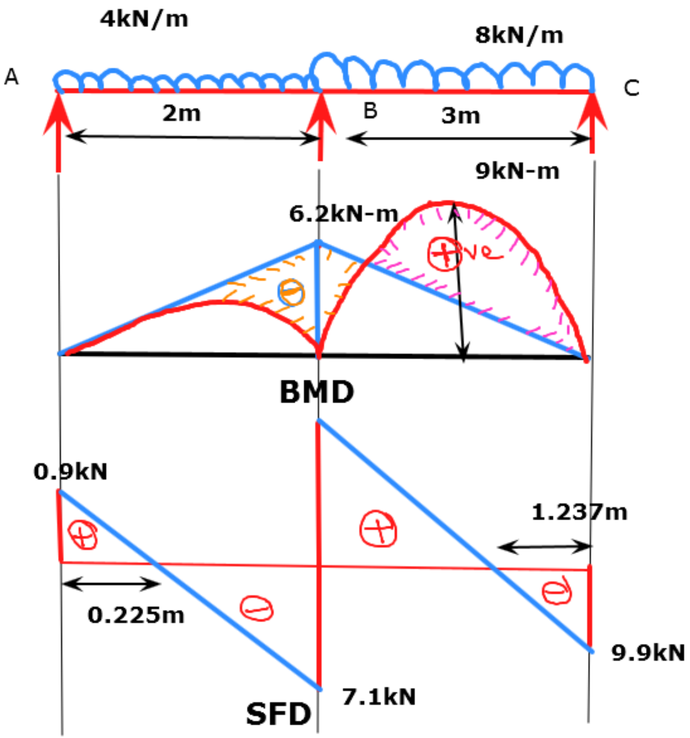
Simply Supported Beam

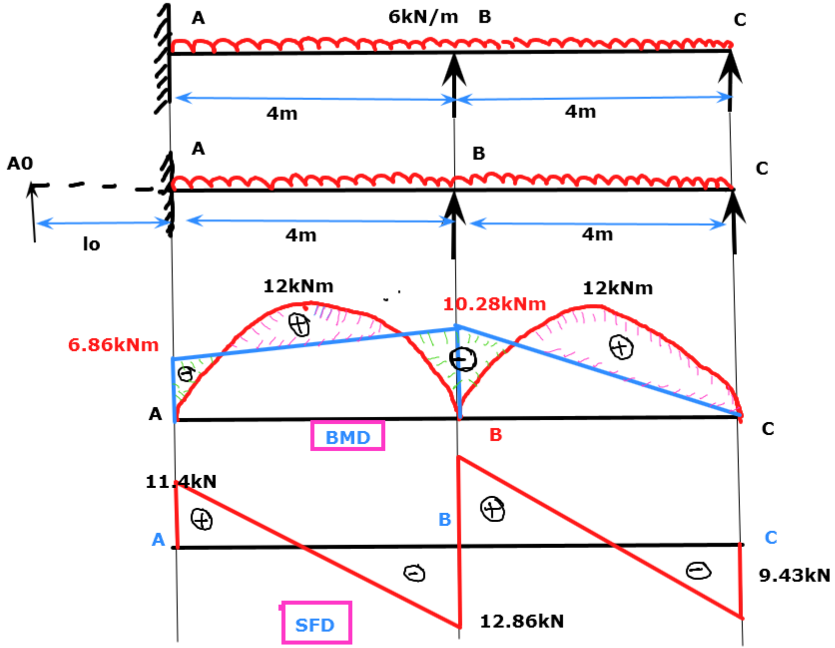




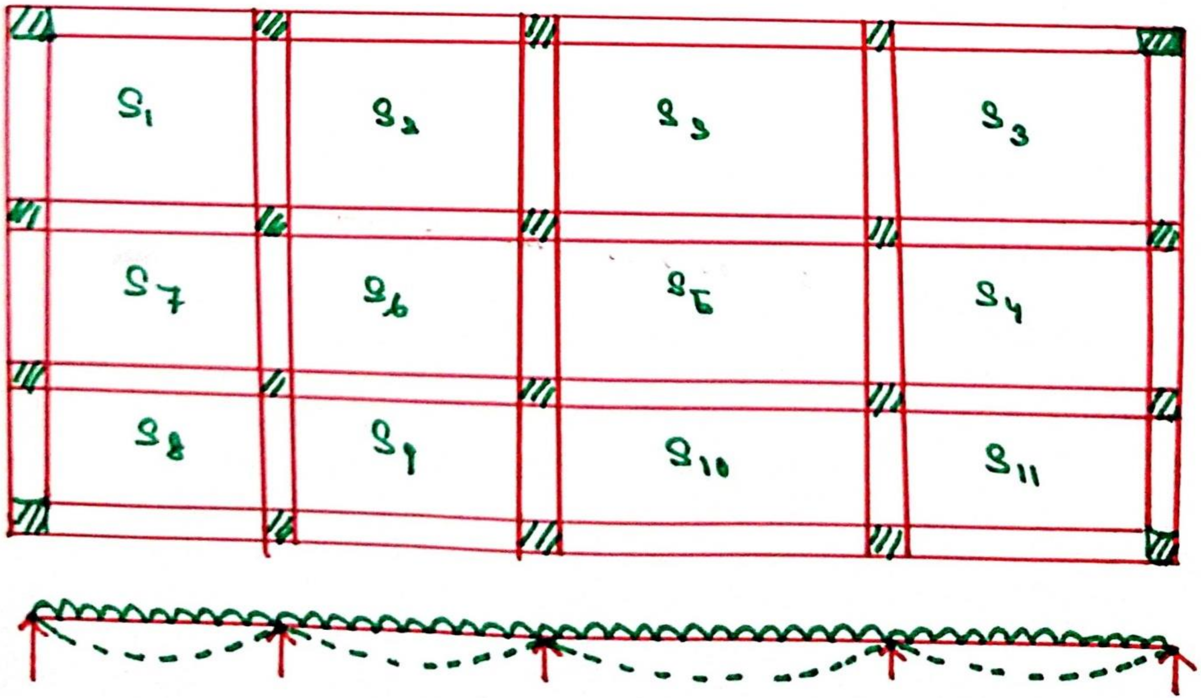
Fixed Beam

CONTINUOUS Beams





TWO WAY CONTINUOUS SLAB



Deflection pattern of the continuous beam

IS:456 2000 Recommendations For Design of Beams

Effective Span: Cl 22.2 IS:456-2000 Pg-34

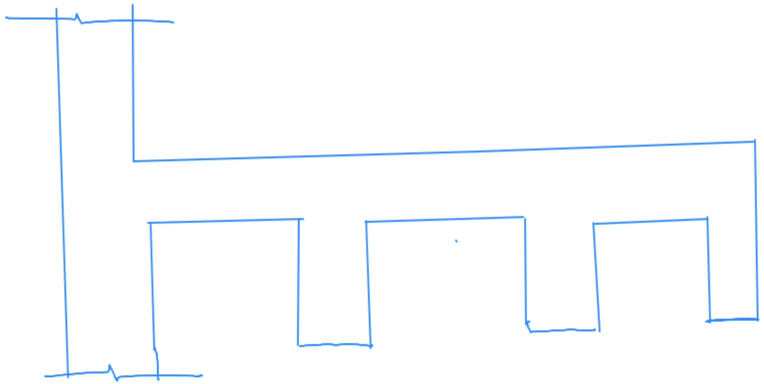
22.2 Effective Span

Unless otherwise specified, the effective span of a member shall be as follows:

a) *Simply Supported Beam or Slab*—The effective span of a member that is not built integrally with its supports shall be taken as clear span plus the effective depth of slab or beam or centre to centre of supports, whichever is less.

34

Effective Span: Cl 22.2 IS:456-2000 Pg-34



b) *Continuous Beam or Slab* — In the case of continuous beam or slab, if the width of the support is less than 1/12 of the clear span, the effective span shall be as in **22.2** (a). If the supports are wider than 1/12 of the clear span or 600 mm whichever is less, the effective span shall be taken as under:

1) For end span with one end fixed and the other continuous or for intermediate spans, the effective span shall be the clear span between supports;

2) For end span with one end free and the other continuous, the effective span shall be equal to the clear span plus half the effective depth of the beam or slab or the clear span plus half the width of the discontinuous support, whichever is less;

3) In the case of spans with roller or rocket bearings, the effective span shall always be the distance between the centres of bearings.

c) *Cantilever* — The effective length of a cantilever shall be taken as its length to the face of the support plus half the effective depth except where it forms the end of a continuous beam where the length to the centre of support shall be taken.

d) *Frames* — In the analysis of a continuous frame, centre to centre distance shall be used.

Reinforcement In Beam

Minimum & Maximum Reinforcement: Cl 26.5.2.1 IS:456-2000 Pg-48

given by the following:

$$\boxed{\frac{A_s}{bd} = \frac{0.85}{f_y}}$$

where

A_s = minimum area of tension reinforcement,

b = breadth of beam or the breadth of the web of T-beam,

d = effective depth, and

f_y = characteristic strength of reinforcement in N/mm².

b) *Maximum reinforcement*—The maximum area of tension reinforcement shall not exceed $\boxed{0.04\ bD.}$

Maximum Spacing of Reinforcement: Cl 26.3.3 IS:456-2000 Pg-46

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.

COVER: Nominal Cover for slab is taken from Table-16, IS:456-2000 Pg-47

IS 456 : 2000

Table 16 Nominal Cover to Meet Durability Requirements

(*Clause* 26.4.2)

Exposure	Nominal Concrete Cover in mm not Less Than
Mild	20
Moderate	30
Severe	45
Very severe	50
Extreme	75

NOTES

1 For main reinforcement up to 12 mm diameter bar for mild exposure the nominal cover may be reduced by 5 mm.

2 Unless specified otherwise, actual concrete cover should not deviate from the required nominal cover by $^{+10}_{0}$ mm

3 For exposure condition 'severe' and 'very severe', reduction of 5 mm may be made, where concrete grade is M35 and above.

Control of Deflection In Beams

Vertical Deflection Limit: Cl 23.2.1 IS:456-2000 Pg-37

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.

MOMENTS IN TWO WAY SIMPLY SUPPORTED RCC SLAB

after bending.

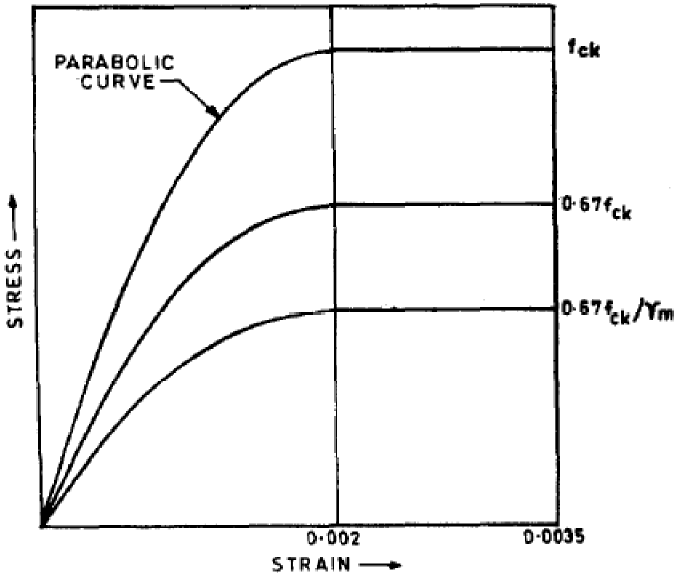


FIG. 21 STRESS-STRAIN CURVE FOR CONCRETE

$\overline{1.15 E_s} + 0.002$

where

f_y = characteristic strength of steel, and
E_s = modulus of elasticity of steel.

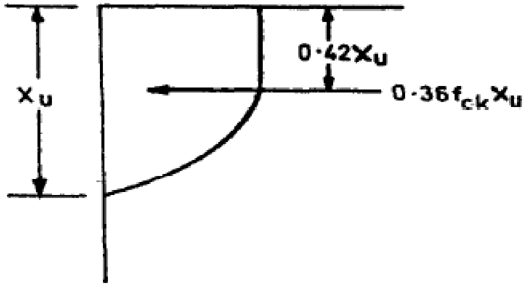
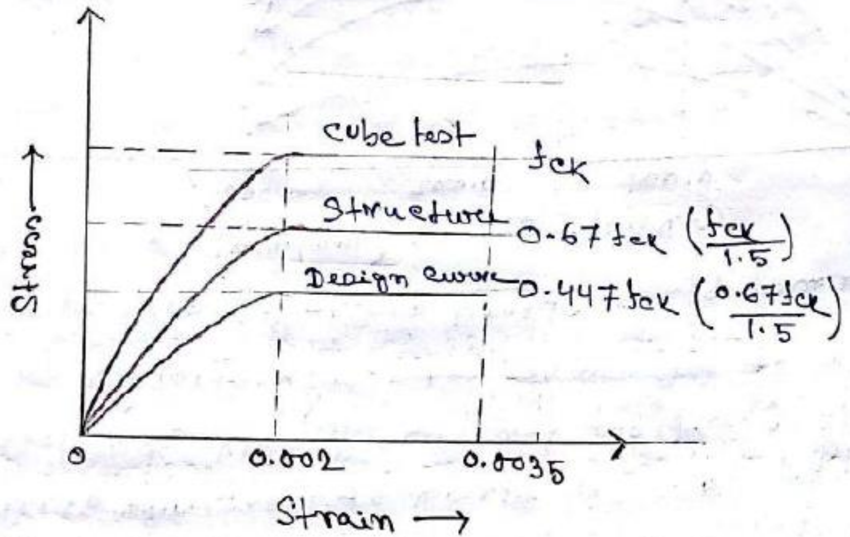


FIG. 22 STRESS BLOCK PARAMETERS

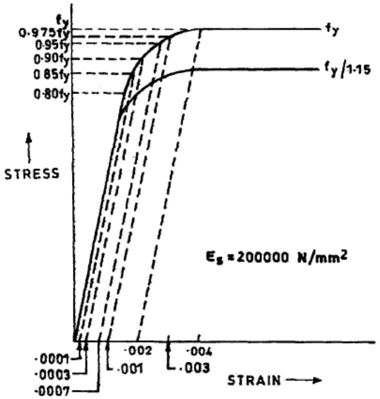
Stress ⟶

Ide

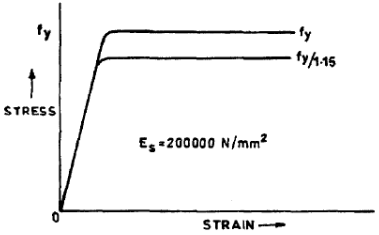
(IS:456-2000; Pg-69)



Idealized Stress Strain Curve for concrete



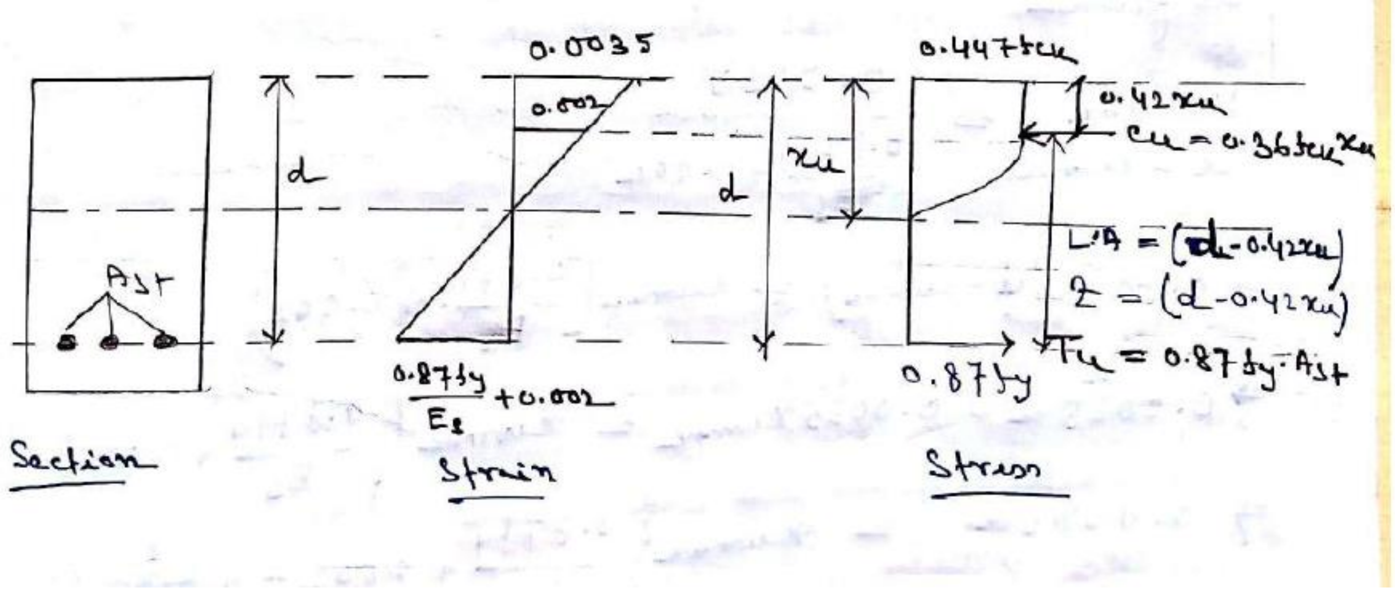
23A Cold Worked Deformed Bar



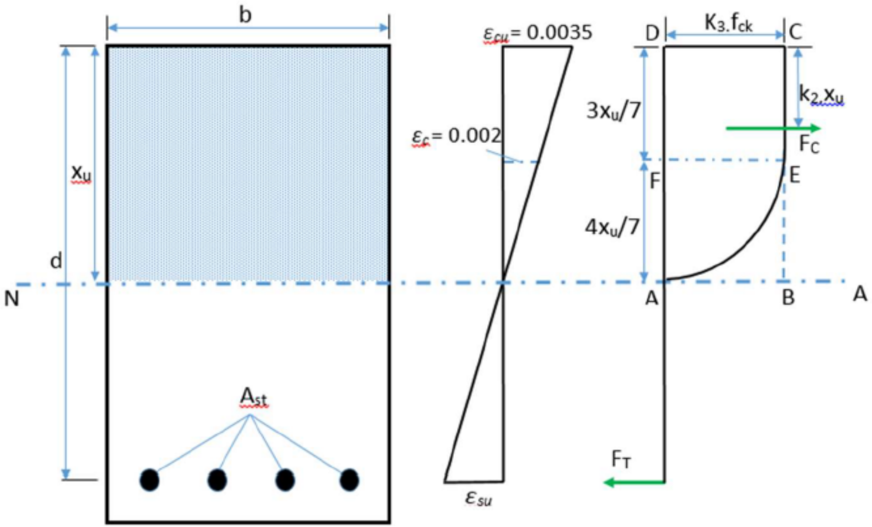
23B STEEL BAR WITH DEFINITE YIELD POINT

FIG. 23 REPRESENTATIVE STRESS-STRAIN CURVES FOR REINFORCEMENT

Stresses and forces in RCC Beam Section



Design Against Bending:



Determine the depth of neutral axis –

$$\frac{x_u}{d} = \frac{0.87}{0.36}\frac{f_y}{f_{ck}}\frac{A_{st}}{bd}$$

Capacity for under reinforced section –

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

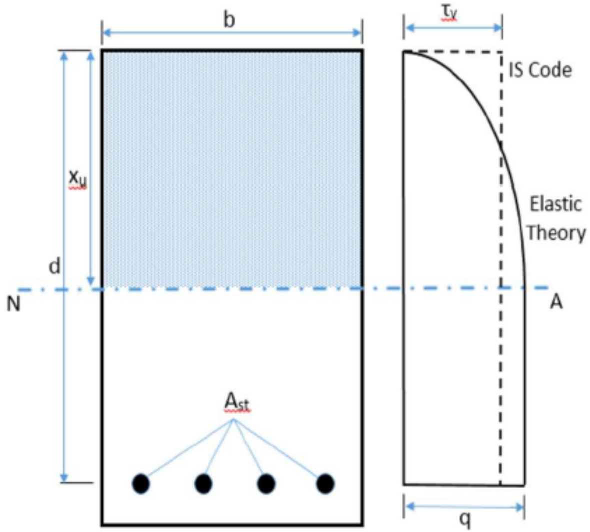
Capacity for balanced section –

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

Caution: Avoid over-reinforced section

Important: (Cl26.2.3.4) Negative moment reinforcement – At least one-third of the reinforcement provided for negative moment at the support shall extent beyond the point of inflection for a distance not less than the effective depth of the member of 12ϕ or one-sixteenth of the clear span whichever is greater.

Design Against Shear:



(Cl40) Limit State of Collapse in Shear for uniform cross section

$$\tau_v = \frac{V_u}{bd}$$

Note: Design shear strength of concrete is given in Table 19

(Cl40) Limit State of Collapse in Shear for non-uniform cross section

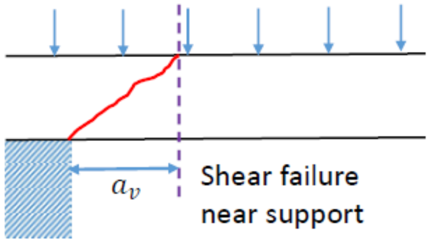
$$\tau_v = \frac{V_u + \frac{M_u}{d} tan(\beta)}{bd}$$

Prefer to avoid in G+2 residential type

(Cl26.5.1.5) Minimum Shear reinforcement

$$\frac{A_{sv}}{bs_v} \geq \frac{0.4}{0.87 f_y}$$

(Cl26.5.1.5) Maximum spacing of shear reinforcement – The maximum spacing along the axis of the member shall not exceed 0.75d for vertical stirrups and d for inclined stirrup at 45 degree



Shear failure near support

(Cl40.2.3) Nominal shear stress – Under no circumstances, even with shear reinforcement, shall the nominal shear stress in beams τ_v exceed τ_{cmax} given in Table 20

Design Against Shear:

40 LIMIT STATE OF COLLAPSE : SHEAR

40.1 Nominal Shear Stress

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

$$\tau_v = \frac{V_u}{b_d}$$

where

V_u = shear force due to design loads;

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

d = effective depth.

40.3 Minimum Shear Reinforcement

When τ_v is less than τ_c given in Table 19, minimum shear reinforcement shall be provided in accordance with **26.5.1.6.**

40.4 Design of Shear Reinforcement

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

a) Vertical stirrups,

b) Bent-up bars along with stirrups, and

Design Against Shear:

Table 19 Design Shear Strength of Concrete, τ_c, N/mm²

(Clauses 40.2.1, 40.2.2, 40.3, 40.4, 40.5.3, 41.3.2, 41.3.3 and 41.4.3)

$100\frac{A_s}{bd}$	Concrete Grade					
	M 15	M 20	M 25	M 30	M 35	M 40 and above
(1)	(2)	(3)	(4)	(5)	(6)	(7)
≤ 0.15	0.28	0.28	0.29	0.29	0.29	0.30
0.25	0.35	0.36	0.36	0.37	0.37	0.38
0.50	0.46	0.48	0.49	0.50	0.50	0.51
0.75	0.54	0.56	0.57	0.59	0.59	0.60
1.00	0.60	0.62	0.64	0.66	0.67	0.68
1.25	0.64	0.67	0.70	0.71	0.73	0.74
1.50	0.68	0.72	0.74	0.76	0.78	0.79
1.75	0.71	0.75	0.78	0.80	0.82	0.84
2.00	0.71	0.79	0.82	0.84	0.86	0.88
2.25	0.71	0.81	0.85	0.88	0.90	0.92
2.50	0.71	0.82	0.88	0.91	0.93	0.95
2.75	0.71	0.82	0.90	0.94	0.96	0.98
3.00 and above	0.71	0.82	0.92	0.96	0.99	1.01

NOTE — The term A_s is the area of longitudinal tension reinforcement which continues at least one effective depth beyond the section being considered except at support where the full area of tension reinforcement may be used provided the detailing conforms to 26.2.2 and 26.2.3

Table 20 Maximum Shear Stress, $\tau_{c\,max}$, N/mm²

(Clauses 40.2.3, 40.2.3.1, 40.5.1 and 41.3.1)

Concrete Grade	M 15	M 20	M 25	M 30	M 35	M 40 and above
$\tau_{c\,max}$, N/mm²	2.5	2.8	3.1	3.5	3.7	4.0

$p_t = 0.955 \simeq 1$

Corresponding $\tau_c = 0.62$ N/mm² [From IS-456, Table-19, Pg-73]

Shear force carried by Concrete $V_{uc} = \tau_c \times (bd)$

$= 0.62 \times (300 \times 535)$ N

$= 99.51 \times 10^3$ N

$= 99.51$ kN

Shear force carried by Steel (Shear reinforcement)

$V_{us} = (V_u - V_{uc})$

$= (157.5 - 99.51) = 57.99$ kN

Let provide two legged vertical stirrups of 8φ bars.

Area of two legged vertical stirrups

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

$= 100$ mm²

Design Against Shear:

a) For vertical stirrups:

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

b) For inclined stirrups or a series of bars bent-up at different cross-sections:

$$V_{us} = \frac{0.8.7 f_y A_{sv} d}{s_v} (\sin\alpha + \cos\alpha)$$

c) For single bar or single group of parallel bars, all bent-up at the same cross-section:

26.5.1.5 *Maximum spacing of shear reinforcement*

The maximum spacing of shear reinforcement measured along the axis of the member shall not exceed 0.75 *d* for vertical stirrups and *d* for inclined stirrups at 45°, where *d* is the effective depth of the section under consideration. In no case shall the spacing exceed 300 mm.

26.5.1.6 ***Minimum shear reinforcement***

Minimum shear reinforcement in the form of stirrups shall be provided such that:

$$\frac{A_{sv}}{b s_v} \geq \frac{0.4}{0.87 f_y}$$

where

A_{sv} = total cross-sectional area of stirrup legs effective in shear,

s_v = stirrup spacing along the length of the member,

b = breadth of the beam or breadth of the

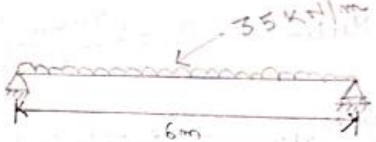
"Design of Singly reinforced Beam" (Part-I) ①

Design a Simply Supported rectangular beam of effective length of 6 m. The beam is subjected to a total load of 35 kN/m (including Self wt). Use M-20 grade Concrete and Fe-415 grade Steel. Using LSM.

Step ⇒ 1: Design Content :⇒

$W = 35\ kN/m$, $L = 6m$

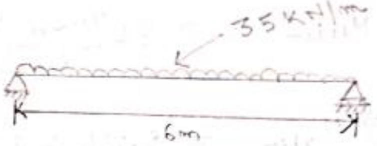
$f_{ck} = 20\ N/mm^2$ $f_y = 415\ N/mm^2$

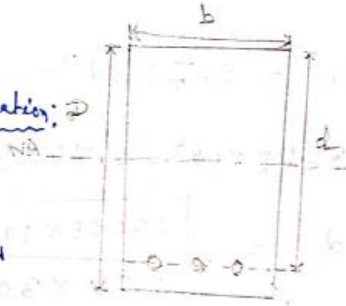


Step → 1 : Design Content : →

$W = 35\ KN/m,\ L = 6m$

$f_{ck} = 20\ N/mm^2 \quad f_y = 415\ N/mm^2$





Step → 2 : Load, SF and BM Calculation : →

Maximum Shear force

$$V_{max} = \frac{WL}{2} = \frac{35 \times 6}{2} = 105\ KN$$

Design Shear force

$$V_u = (1.5 \times 105)\ KN = 157.5\ KN$$

$F_d = \gamma_f \times F$

partial Safety factor

Load

design Load

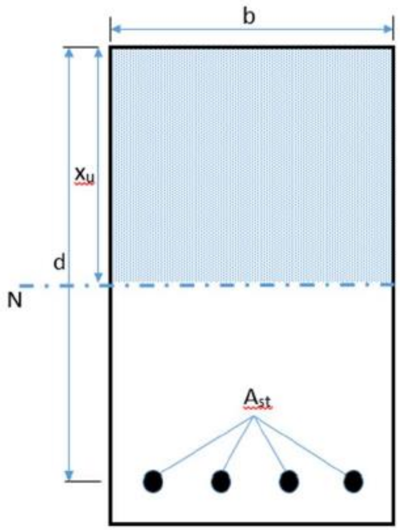
Maximum bending moment

$$M_{max} = \frac{WL^2}{8} = \frac{35 \times 6^2}{8} = 157.5\ KN\text{-}m$$

Design bending moment $M_u = (1.5 \times 157.5)$

$= 236.25\ KN\text{-}m$

Step → 3

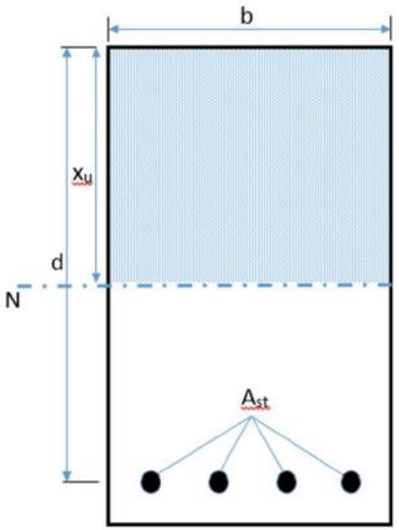


$= 236.25$ KN-m

Step ⇒ 3: Depth Calculation :⇒

Let try to design the beam as a balance section

Assume $b = 300$ mm



From SP-16, Table-D, Pg-10, For M-20 concrete and Fe-415 steel

$$\frac{M_u}{bd^2} = 2.76$$

$$\Rightarrow M_u = 2.76\, bd^2$$

or

Use

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

IS-456:2000
Pg-96
cl-G.1.1

$\frac{x_{umax}}{d} = 0.48$

Pg-70
IS-456

$$\Rightarrow M_{ulim} = 2.76\, bd^2$$

$$\Rightarrow M_u = 2.76\, bd^2$$

$$\Rightarrow 236.25 \times 10^6 = 2.76 \times 300 \times d^2$$

$$\Rightarrow d = \sqrt{\frac{236.25 \times 10^6}{2.76 \times 300}} = 534.159 \text{ mm}$$

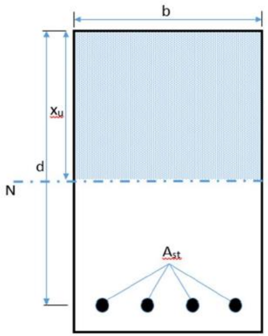
$d_{req} = 534.159$ mm

$d_{prov} = 535$ mm

(iii) Step → 3: Depth Calculation :→

Let try to design the beam as a balance section

Assume $b = 300\ mm$



Let provide $20\ \phi$ dia bars and a clear cover $= 20\ mm$ (Mild exposure)

{IS-456:2000, Table-16, Pg - 47}

$$D_{prov} = \left(535 + 20 + \frac{20}{2}\right) = 565\ mm$$

From SP-16, Table-D, Pg-10, For M-20 concrete and Fe-415 steel

$$\frac{M_u}{bd^2} = 2.76$$

$$\Rightarrow M_u = 2.76\ bd^2$$

or

use

$$M_{ulim} = 0.36\frac{x_{umax}}{d}\left(1 - 0.42\frac{x_{umax}}{d}\right) f_{ck}\, b d^2$$

IS-456:2000, Pg-96, cl-G.1.1

$\frac{x_{umax}}{d} = 0.48$ Pg-70 IS-456

$$\Rightarrow M_{ulim} = 2.76\ bd^2$$

$$\Rightarrow M_u = 2.76\ bd^2$$

$$\Rightarrow 236.25 \times 10^6 = 2.76 \times 300 \times d^2$$

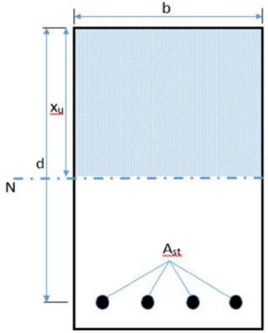
$$\Rightarrow d = \sqrt{\frac{236.25 \times 10^6}{2.76 \times 300}} = 534.159\ mm$$

$d_{req} = 534.159\ mm$

$d_{prov} = 535\ mm$

Step → 4: Area of Steel Calculation :→

$$\frac{M_u}{bd^2} = \frac{236.25 \times 10^6}{300 \times 535^2} = 2.75 \simeq 2.76$$



Let provide 20 ϕ dia bars and clear cover = 20mm (Mild exposure)

{IS-456:2000, Table-16, Pg - 47}

$$D_{prov} = \left(535 + 20 + \frac{20}{2}\right) = 565\,mm$$

Now from SP-16, Page → 48

$p_t = 0.955$ for corresponding value of $\frac{M_u}{bd^2} = 2.76$

$$\Rightarrow p_t = \frac{A_{st} \times 100}{bd} = 0.955$$

$$\Rightarrow A_{st} = \left(\frac{0.955 \times 300 \times 535}{100}\right) = 1532.775\,mm^2$$

Area of 20 ϕ bars $= \frac{\pi}{4} \times 20^2 = 314.15\,mm^2$

No of bars required $= \frac{1532.775}{314.15} = 4.87$

$\simeq$ 5 nos

TABLE 2 FLEXURE — REINFORCEMENT PERCENTAGE, p_t FOR SINGLY REINFORCED SECTIONS

f_{ck} = 20 N/mm²

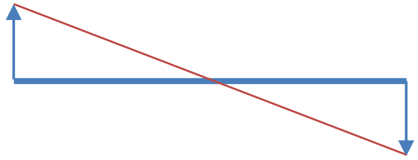
M_u/bd^2, N/mm²	f_y, N/mm² 240	250	415	480	500
0·30	0·146	0·140	0·085	0·073	0·070
0·35	0·171	0·164	0·099	0·086	0·082
0·40	0·196	0·188	0·114	0·098	0·094
0·45	0·222	0·213	0·128	0·111	0·106
0·50	0·247	0·237	0·143	0·123	0·119
0·55	0·272	0·262	0·158	0·136	0·131
0·60	0·298	0·286	0·172	0·149	0·143
0·65	0·324	0·311	0·187	0·162	0·156
0·70	0·350	0·336	0·203	0·175	0·168
0·75	0·376	0·361	0·218	0·188	0·181
0·80	0·403	0·387	0·233	0·201	0·193
0·85	0·430	0·412	0·248	0·215	0·206
0·90	0·456	0·438	0·264	0·228	0·219
0·95	0·483	0·464	0·280	0·242	0·232
1·00	0·511	0·490	0·295	0·255	0·245
1·05	0·538	0·517	0·311	0·269	0·258
1·10	0·566	0·543	0·327	0·283	0·272
1·15	0·594	0·570	0·343	0·297	0·285
1·20	0·622	0·597	0·359	0·311	0·298
1·25	0·650	0·624	0·376	0·325	0·312
1·30	0·678	0·651	0·392	0·339	0·326
1·35	0·707	0·679	0·409	0·354	0·339
1·40	0·736	0·707	0·426	0·368	0·353
1·45	0·765	0·735	0·443	0·383	0·367
1·50	0·795	0·763	0·460	0·397	0·382
1·55	0·825	0·792	0·477	0·412	0·396
1·60	0·855	0·821	0·494	0·427	0·410
1·65	0·885	0·850	0·512	0·443	0·425
1·70	0·916	0·879	0·530	0·458	0·440
1·75	0·947	0·909	0·547	0·473	0·454
1·80	0·978	0·939	0·565	0·489	0·469
1·85	1·009	0·969	0·584	0·505	0·484
1·90	1·041	1·000	0·602	0·521	0·500
1·95	1·073	1·030	0·621	0·537	0·515
2·00	1·106	1·062	0·640	0·553	0·531
2·02	1·119	1·074	0·647	0·559	0·537
2·04	1·132	1·087	0·655	0·566	0·543
2·06	1·145	1·099	0·662	0·573	0·550
2·08	1·159	1·112	0·670	0·579	0·556
2·10	1·172	1·125	0·678	0·586	0·562
2·12	1·185	1·138	0·685	0·593	0·569
2·14	1·199	1·151	0·693	0·599	0·575
2·16	1·212	1·164	0·701	0·606	0·582
2·18	1·226	1·177	0·709	0·613	0·588
2·20	1·239	1·190	0·717	0·620	0·595

M_u/bd^2, N/mm²	f_y, N/mm² 240	250	415	480	500
2·22	1·253	1·203	0·725	0·627	0·602
2·24	1·267	1·216	0·733	0·633	0·608
2·26	1·281	1·230	0·741	0·640	0·615
2·28	1·295	1·243	0·749	0·647	0·621
2·30	1·309	1·256	0·757	0·654	0·628
2·32	1·323	1·270	0·765	0·661	0·635
2·34	1·337	1·283	0·773	0·668	0·642
2·36	1·351	1·297	0·781	0·675	0·648
2·38	1·365	1·311	0·790	0·683	0·655
2·40	1·380	1·324	0·798	0·690	0·662
2·42	1·394	1·338	0·806	0·697	0·669
2·44	1·408	1·352	0·814	0·704	0·676
2·46	1·423	1·366	0·823	0·711	0·683
2·48	1·438	1·380	0·831	0·719	0·690
2·50	1·452	1·394	0·840	0·726	0·697
2·52	1·467	1·408	0·848	0·734	0·704
2·54	1·482	1·423	0·857	0·741	0·711
2·56	1·497	1·437	0·866	0·748	0·719
2·58	1·512	1·451	0·874	0·756	0·726
2·60	1·527	1·466	0·883	0·764	0·733
2·62	1·542	1·481	0·892	0·771	0·740
2·64	1·558	1·495	0·901	0·779	0·748
2·66	1·573	1·510	0·910	0·786	0·755
2·68	1·588	1·525	0·919	0·794	
2·70	1·604	1·540	0·928		
2·72	1·620	1·555	0·937		
2·74	1·636	1·570	0·946		
2·76	1·651	1·585	0·955		
2·78	1·667	1·601			
2·80	1·683	1·616			
2·82	1·700	1·632			
2·84	1·716	1·647			
2·86	1·732	1·663			
2·88	1·749	1·679			
2·90	1·766	1·695			
2·92	1·782	1·711			
2·94	1·799	1·727			
2·96	1·816	1·743			
2·98	1·833	1·760			

NOTE — Blanks indicate inadmissible reinforcement percentage (*see* Table E).

Step ⇒ 5 : Design against Shear :

Design Shear force $V_u = 157.5$ kN



SFD

$p_t = 0.955 \simeq 1$

Corresponding $\tau_c = 0.62$ N/mm² [From IS-456, Table-19, Pg-73]

Shear force Carried by Concrete $V_{uc} = \tau_c \times (b\,d)$

$= 0.62 \times (300 \times 535)$ N

$= 99.51 \times 10^3$ N

$= 99.51$ kN

Shear force Carried by Steel (Shear reinforcement)

$V_{us} = (V_u - V_{uc})$

$= (157.5 - 99.51) = 57.99$ kN

Let provide two leged vertical Stirrups of 8 ϕ bars.

Area of two leged vertical Stirrups

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

$= 100$ mm²

Step ⇒ 5 : Design against Shear :

Design Shear force $V_u = 157.5\ kN$

(a) $= 100\ mm^2$

Now from IS:456-2000, cl-40.4.a, pg-73 for vertical

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

S_v = Spacing of Stirrups

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

$$= \frac{0.87 \times 415 \times 100 \times 535}{57.99 \times 10^3} = 333.09\ mm$$

(b) Maximum Spacing Criteria :

From cl-26.5.1.5, IS-456:2000, Pg-47

Maximum Spacing of Shear reinforcement

$$= \min\{0.75d, 300\}$$

$$= \min\{401.25, 300\}$$

$$= 300\ mm$$

(c) Minimum Shear reinforcement :

From cl-26.5.1.6, IS-456:2000

$$\frac{A_{sv}}{b S_v} \geq \frac{0.4}{0.87 f_y}$$

$$\Rightarrow S_v \leq \frac{0.87 f_y A_{sv}}{0.4 b}$$

$$\Rightarrow S_v \leq 300.87\ mm$$

Let provide two leged vertical Stirrups of 8 ϕ bars @ 300 c/c.

Step → 6 : Checks ⇒

i) Shear check :→

From cl-40.1, IS-456:2000, Pg-72

Nominal Shear Stress

$$\tau_v = \frac{V_u}{bd}$$

$$= \frac{157.5 \times 10^3}{300 \times 535} = 0.981 \ N/mm^2$$

$p_t = 1$ From Table-19, IS-456:2000

Corresponding τ_c

$\tau_c = 0.62 \ N/mm^2$

From Table-20, IS-456:2000, Pg-73

Maximum Shear Stress for M-20 concrete

$\tau_{c max} = 2.8$

$\tau_{c max} > \tau_v$ Hence design is safe

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

ii) Deflection check :⇒

$$\frac{Span}{d} = \frac{6 \times 10^3}{535} = 11.21$$

Now from cl-23.2.1, IS-456:2000, Pg-37

$\frac{Span}{d} = 20$ for simply supported beam

∴ 11.21 < 20 Hence the design is safe.

$\left(\frac{L}{d}\right)_{provided} < \left(\frac{L}{d}\right)_{allowable}$

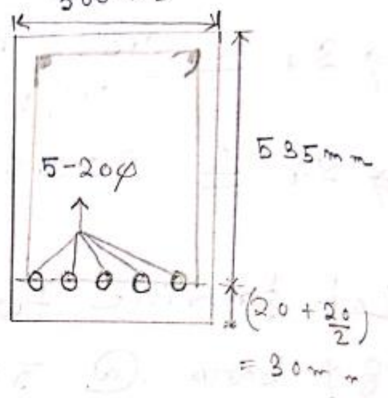
(11) Summary of the design :⇒

Beam Size (b×D) ⇒ (300 × 565) mm

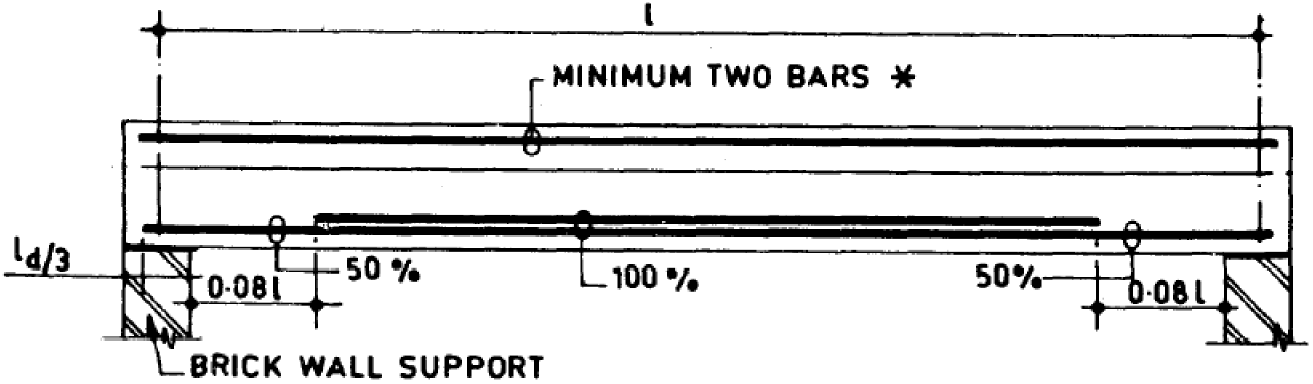
Clear cover ⇒ 20 mm

Tensile Steel ⇒ 20 φ – 5 nos

Stirrups ⇒ 8 φ two leged vertical stirrups of @ 300 c/c



DETAILING



*In case partially restraint members, 35 percent of the reinforcement shall also be provided for negative moment at the support and fully anchored.

FIG. 8.16 SIMPLIFIED CURTAILMENT RULES FOR SIMPLY SUPPORTED BEAM

DETAILING

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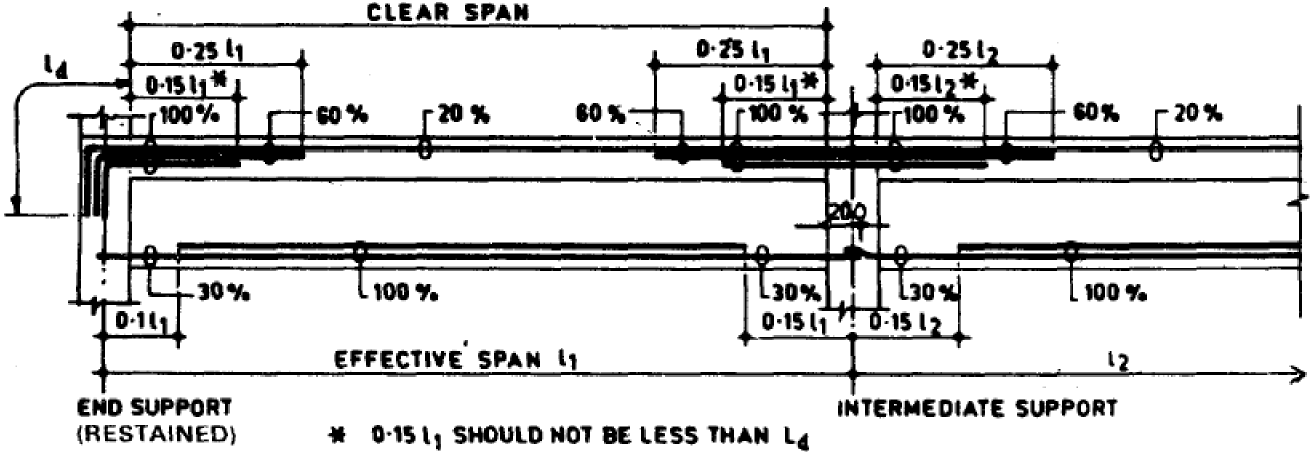
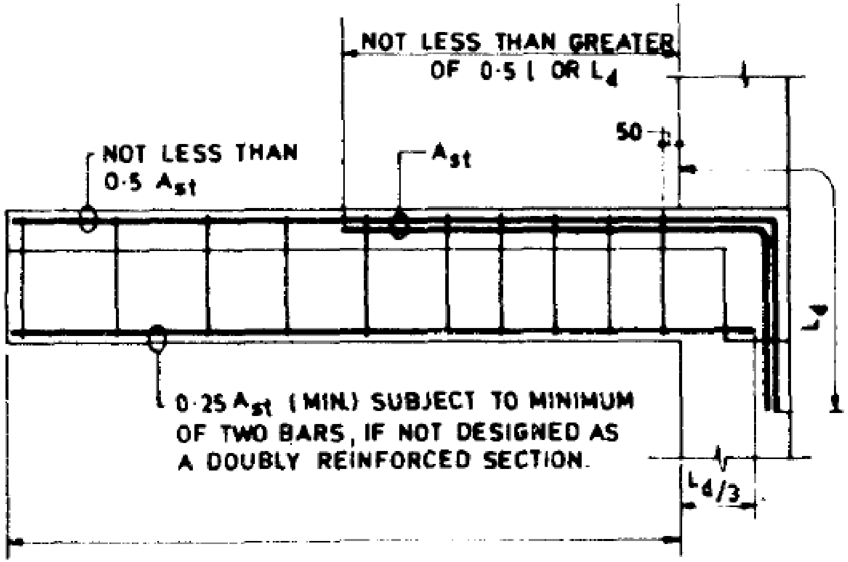


NOTE — Applicable to continuous beams with approximately equal spans (not differing more than 15 percent) and subjected to predominantly U.D.L., and designed without compression steel.

FIG. 8.15 SIMPLIFIED CURTAILMENT RULES FOR CONTINUOUS BEAMS

DETAILING



8.17A CANTILEVER BEAM PROJECTING FROM A COLUMN

FIG. 8.17 SIMPLIFIED CURTAILMENT RULES FOR A CANTILEVER BEAM (*Continued*)

DETAILING

SP : 34(S&T)-1987

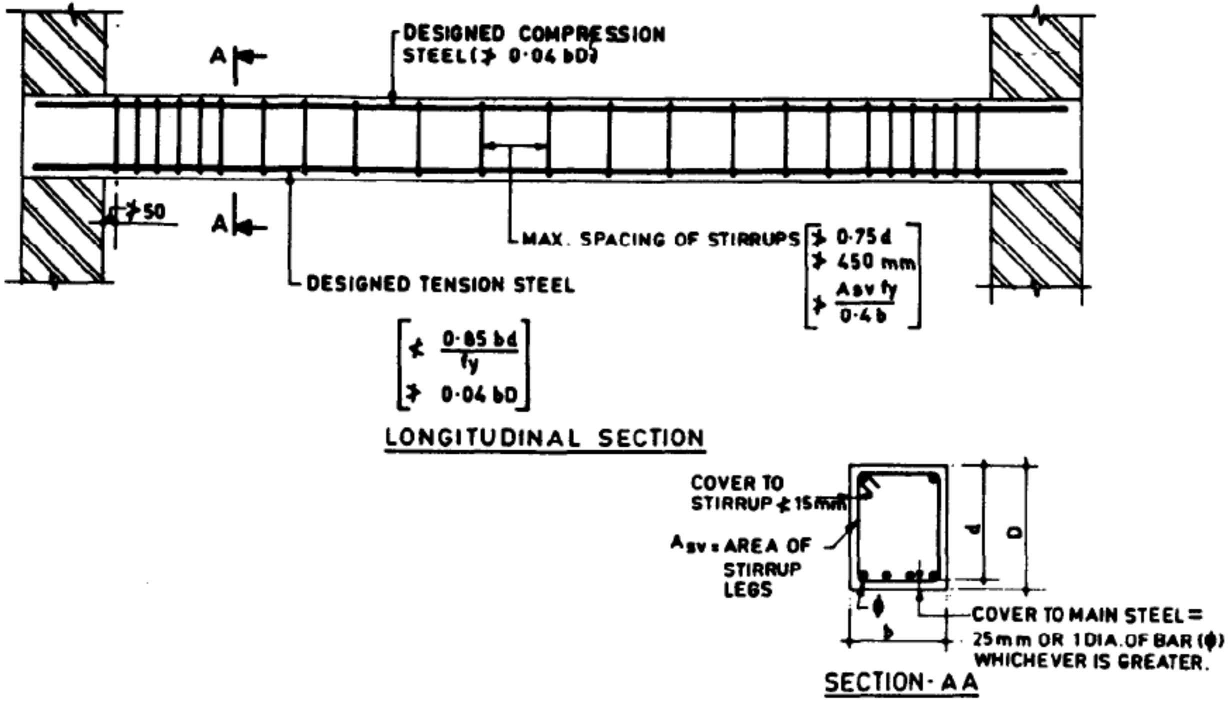


FIG. 8.11 REINFORCEMENT REQUIREMENTS FOR BEAMS

THANK YOU