UNIT 11 DESIGN AND DETAILING OF BEAMS & LINTELS

Structure

11.1 INTRODUCTION

Design of a beam or a lintel means fixing the size of concrete section, determining the areas of tensile as well as compressive (if necessary) reinforcements, provision of shear and torsion reinforcements and curtailment of tensile reinforcements. Analysis for design is based on design (actual) load. The safety and serviceability is ensured by keeping stresses deflection and cracking below the permissible limits. The detailing rules enunciated in the code ensures complex, unevalulated or semi empirical stresses below the allowable limits.

Objectives

After going through the text and illustrations of this unit, you should be able to design a simply supported beam, a cantilever beam, a continuous beam, and a lintel.

11.2 TYPES OF BEAMS AND THEIR DESIGN

From analysis and detailing point of view, a beam may be of one of the following types:

(a) Simply supported beam

(b) Cantilever beam

(c) Continuous beam, and

(d) Overhanging beam

For the design of an overhanging beam, the detailings of overhang portion can be done as a cantilever beam and that of supported side as at intermediate support of a continuous beam. Hence, only first three types of beams have been illustrated here.

Example 11.1

Design a simply supported rectangular beam of clear span 6m and supported on 375 thick walls. The beam is loaded with a dead load of 16 kNm and a live load of 12 kNm. Use M 15 concrete and Fe 250 steel.

Solution

Depth (D)

(i) *From Thumb Rule*

D lying between $\frac{l_{ef}}{10}$ and $\frac{l}{20}$

Assuming $\frac{l_{ef}}{D} = 10$ and taking $l_{ef} \approx 6 + 0.375 = 6.375$ m

or $$D \geq \frac{6.375 \times 10^3}{10} = 638 \approx 700$$

Accordingly $D = 700$

and b (taken between $\frac{1}{3}$rd to $\frac{2}{3}$rd of D) = 400

(ii) *From control of deflection criteria*

$$d > \frac{l_{ef}}{K_B K_1 K_2 K_3}$$

$l_{ef} = 6 + 0.375 = 6.375$ m at the outset as effective depth is not known

$K_B = 20$

K_1 for 0.714% balanced steel = 1.68

$K_2 = K_3 = 1$

$$d > \frac{6.375 \times 10^3}{20 \times 1.68 \times 1 \times 1} = 190$$

Calculation of Loads

Self $= 0.70 \times 0.40 \times 1 \times 25$		=	7.00 kNm
DL		=	16.00 kNm
	Total *DL*	=	23.00 kNm
LL		=	12.00 kNm
	Total Loads	=	35.00 kNm

Maximum B.M., $M = \frac{w l_{ef}^2}{8} = \frac{35 \times 6.375^2}{9} = 170.80$ kNm

$$d = \sqrt{\frac{M}{R_B b}} = \sqrt{\frac{178.8 \times 10^6}{0.867 \times 400}} = 716.03$$

Taking steel in two layers and keeping a clear vertical spacing of 25 mm in between the layers,

$D = 716.03 + 25 + 20 + 12.5 = 773.53 \approx 775$

Hence, provided $D = 775$ and $d = 775 - 25 - 20 - 12.5 = 717.5$

$$\therefore l_{ef} = 6 + 0.375\text{m} = 6.375\text{m}$$

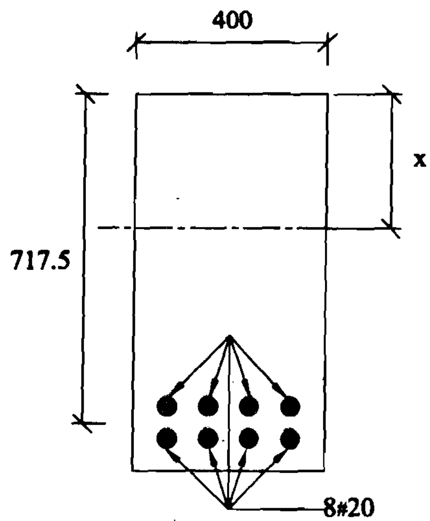

Figure 11.1 : Designed Section of the Beam

Check for b

$l_c = 6000 < 60$ b

or $b > 100$

Again

$$l_c < \frac{250 \times b^2}{717.5}$$

or $$b > \sqrt{\frac{6000 \times 717.5}{250}}$$

or $b > 131.23$

Hence provided $b = 400$

Provision of Reinforcement

$$A_{st} = \frac{M}{\sigma_{st} j_B d} = \frac{177.8 \times 10^6}{140 \times 0.867 \times 717.5} = 2041.56 \text{ mm}^2$$

Provided 8ϕ20 in two layers ($A_{st} = 2512$ mm^2)

Evaluation of Moment of Resistance of the Section

Taking moment of area about n.a.

$$b\frac{x^2}{2} = m\text{A}_{st}(d - x)$$

or $$\frac{400}{2}x^2 = 19 \times 2512\,(d - x)$$

or $$x^2 + \frac{19 \times 2512}{200}x - \frac{19 \times 2512 \times 717.5}{200} = 0$$

or $x^2 + 238.64x - 171224.2 = 0$

or $x = \frac{-238.64 \pm \sqrt{56949.05 + 684896.8}}{2}$

or $x = \frac{-238.64 \pm 861.30}{2} = 311.33$

$x_B = 0.404 \times 717.5 = 289.87 < 311.33$

Hence the section is over reinforced.

$$M_R = \frac{1}{2} \times 5 \times 400 \times 311.33 \times \left(717.5 - \frac{311.33}{3}\right)$$

$$= 191.07 \text{ kNm} > 177.80 \text{ kN. Hence, O.K.}$$

Check

Loads

Self	$0.775 \times 0.4 \times 1 \times 25$	= 7.75 kN/m
DL		= 16.00 kN/m
	Total *DL*	= 23.75 kN/m
LL		= 12.00 kN/m
	Total *DL* + *LL*	= 35.75 kN/m

$$M_{max} = \frac{wl_{ef}^2}{8} = \frac{35.75 \times 6.375^2}{8} = 181.61 \text{ kN/m} < 191.07 \text{ kN/m}$$

Provision of Shear Reinforcement

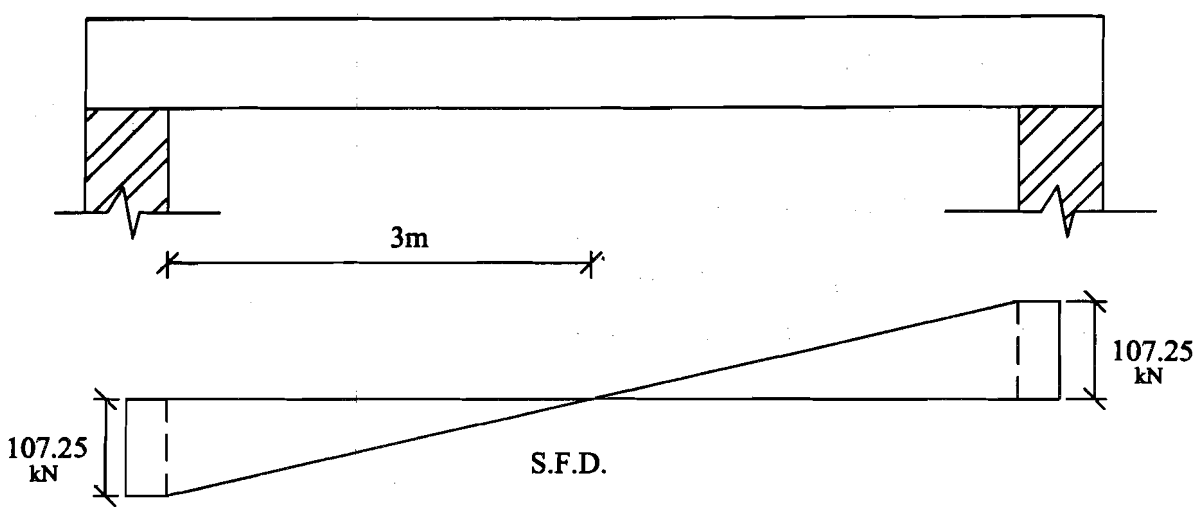

Figure 11.2 : S. F. D. for Design

SF at face of support

$$\frac{wl}{2} = 35.75 \times 3 = 107.25 \text{ kN}$$

$$\tau_v = \frac{V}{bd} = \frac{10.75 \times 10^3}{400 \times 717.5} = 0.374 \text{ N/mm}^2$$

Taking 50% of tensile steel being extended to the support,

$$\frac{100A_s}{bd} = \frac{100 \times \frac{2512}{2}}{400 \times 717.5} = 0.438$$

$$\therefore \tau_c = 0.22 + \frac{(0.29-0.22)}{(0.25)} \times 0.188$$

or $\tau_c = 0.273$ MPa

$\tau_{c,max} = 1.6$ MPa for M 15 concrete

since $\tau_c < \tau_v < \tau_{c,\,max}$

$V_s = V - \tau_c bd = 107.25 - 0.273 \times 400 \times 717.5 \times 10^{-3} = 28.90$ kN

Providing ϕ8 two-legged stirrups

$$s_v = \frac{\sigma_{sv} A_{sv} d}{V_s} = \frac{140 \times 100 \times 717.5}{28.9 \times 10^3} = 349.42$$

Spacing s_v shall also be less than the followings :

(i) $s_v = \dfrac{0.87 f_y A_{sv}}{0.4b} = \dfrac{0.87 \times 250 \times 100}{0.4 \times 400} = 135.94$

(ii) $s_v = 0.75, d = 0.75 \times 717.5 = 538.13$

(iii) $s_v = 450$

Hence provided ϕ8 two-legged vertical stirrups @135

Curtailment of Reinforcement

M_R for 4ϕ20

Let upper 4 bars be curtailed. Then the effective depth for the remaining bars,

$d = 775 - 25 - 10 = 740$

Taking moment of area about n.a. (Figure 11.3)

$$\frac{400}{2}x^2 = 19 \times 4 \times 314 \times (740 - x)$$

or $$x^2 + \frac{19 \times 4 \times 314x}{200} - \frac{19 \times 4 \times 314 \times 740}{200} = 0$$

or $$x^2 + 119.32x - 88296.8 = 0$$

or $$x = \frac{-119.32 \pm \sqrt{14237.26 + 4 \times 88296.8}}{2}$$

or $$x = \frac{-119.32 \pm 606.16}{2} = 243.42$$

$$x_B = 0.404 \times 740 = 298 > 243.42$$

Hence the section is over-reinforced.

$$M_R = \sigma_{st} A_{st}\left(d - \frac{x}{3}\right) = 140 \times 4 \times 314\left(740 \frac{243.42}{3}\right) \times 10^{-6} \text{ kNm}$$

$$= 115.853 \text{ kNm}$$

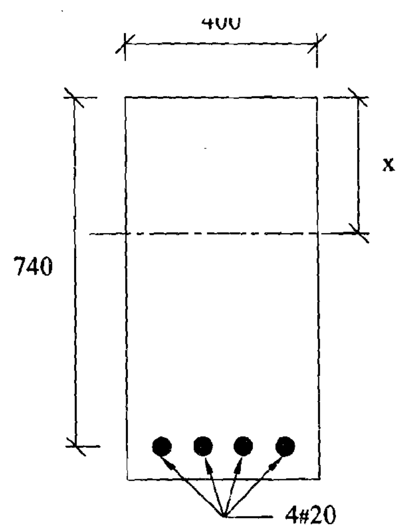

Figure 11.3 : Section of the Beam after Curtailment

Let x' be the distance from support where be above M_R occur, (Figure 11.4)

or $$M_R = \frac{wl_{ef}}{2}x' - \frac{wx'}{2} = \frac{35.75 \times 6.375}{2}x' - \frac{35.75x'}{2}$$

or $$115.853 = 113.95\,x' - 17.875\,x'$$

or $$x' - 6.375\,x' + 6.48 = 0$$

or $$x' = \frac{+6.375 \pm \sqrt{40.64 - 4 \times 6.48}}{2}$$

or $$x' = \frac{+6.375 \pm 3.836}{2} = 1.27\text{m}$$

i.e. theoretical cut-off section from *mid span* $= \frac{6.375}{2} - 1.27 = 1.918\text{m}$

Actual cut off point from *mid span* = 1.918 + (lesser of 12ϕ or d)
= 1.918 + 0.740 = 2.658m

$$L_d = \frac{\phi \times \sigma_s}{4 \times \tau_{bd}} = \frac{20 \times 140 \times 10^{-3}}{4 \times 0.6} = 1.167\text{m} < 2.658\text{m}$$

Length available from the actual cut-off point to the centre line of the support

$= \frac{6.375}{2} - 2.658 = 0.53\text{m} < L_d (1.167\text{m})$

Therefore, portion of L_d to be provided *beyond* centre line of support

$= 1.167 - 0.53 = 0.637$ m (L_0)

Available straight portion of proposed extension beyond centre line of support,

$$x = \frac{\text{Support width}}{2} - \text{(clear cover + dia of bar of bend + radius of bend)}$$

$$= \frac{375}{2} - (25 + \phi + 2\phi) = \frac{375}{2} - (25 + 20 + 2 \times 20) = 102.5$$

Length of straight portion to the provided beyond the bend,

$e =$ Total extension reqd. beyond centre line of support - equivalent length of bend-available straight portion beyond centre line of support

$= 637 - 8\phi - 102.5$

$= 637 - 8 \times 20 - 102.5 = 374.5 \approx 375$

Check

(i) Extension of positive reinforcement into the support

$$> \frac{L_d}{3}\left(= \frac{1167}{3} = 389\right) < 637. \text{ Hence } \quad \text{O.K.}$$

(ii) At support $L_d \leq 1.3\frac{M_1}{V} + L_0$

where L_d = 1.167 m

M_1 = 115.853 kNm

and V = S.F. at simple support due to design load

= 35.75 × 6.375/2 = 113.95 kN

and L_0 = 0.637 m

Substituting these values in the above equation

$$1.167 \leq 1.3\frac{115.853}{113.95} + 0.637$$

$$\leq 1.96 \text{ m Hence O.K.}$$

Provision of stirrups in excess at cut-off point

As per Code (cl 25.2.3. 2*b*) distance *beyond* cut-off point upto which shear stirrups in excess to be provided = $\frac{3}{4}d = \frac{3}{4} \times 740 = 555 > 342.5$ (distance of face of support)

Excess area, $A_{sve} = \frac{0.4b_s}{f_y}$

$$A_{sv} + A_{sve} = \frac{V_s s_v}{\sigma_{sv} d} + \frac{0.4bs_v}{f_y}$$

(for ϕ8 - two-legged vertical stirrups, $A_{sv} + A_{sve} = 100$)

or $$100 = \left(\frac{28.9 \times 10^3}{140 \times 740} + \frac{0.4 \times 400}{250}\right)s_v$$

or $s_v = 108.8$

Again $\frac{d}{8\beta} = \frac{740}{8 \times 0.5} = 185$

Hence provided ϕ8 two-legged vertical stirrups @ 105

The details of reinforcement is shown in Figure 11.4

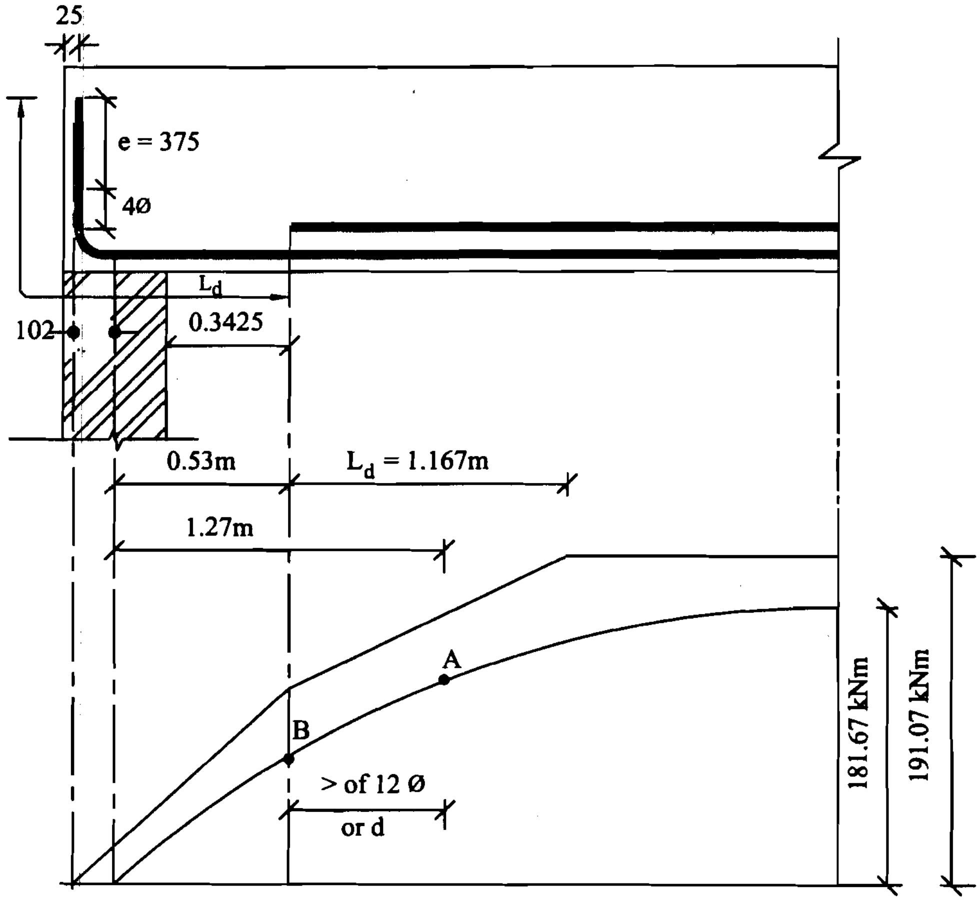

Figure 11.4 : Curtailment and Detailing of the Reinforcement

Example 11.2

Design a cantilever beam shown in Figure 11.5 below. Use M 15 concrete and Fe 250 steel.

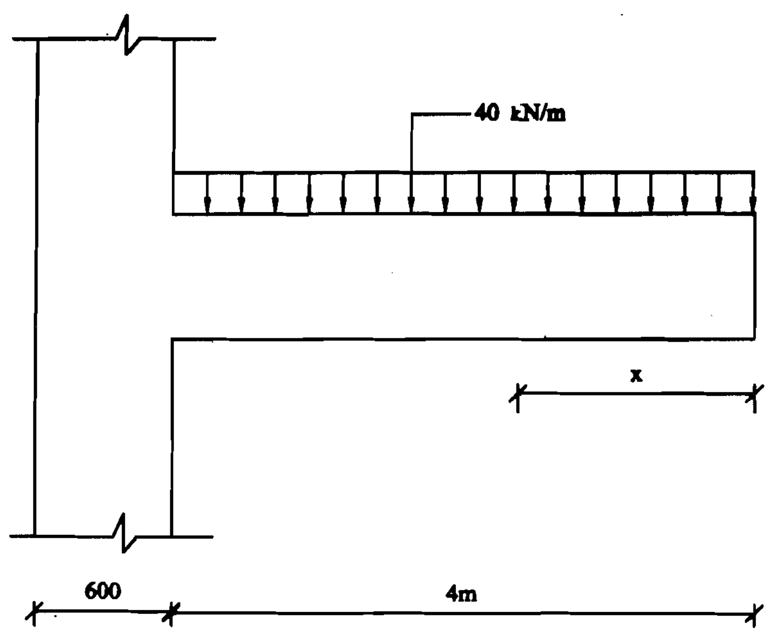

Figure 11.5 : A Cantilever Beam Projecting from a RCC Column

Solution

Effective span

$L_{ef} = 4\text{m}$

Depth (*D*)

(i) *From Deflection Criteria*

$$\frac{l_{ef}}{d} < K_B K_1 K_2 K_3$$

where $K_B = 7$

$K_1 = 1.68$ for 0.714% of balanced steel

$K_2 = K_3 = 1$

$$d > \frac{4 \times 1000}{7 \times 1.68} = 340.136$$

Taking $D = 850$; $b = 400$

and $d = 850 - 50 = 800$

Check for *b*

$$l_{ef} < 25\, b$$

or $$b > \frac{4000}{25} \; (=160)$$

Again $$l_{ef} < \frac{100b^2}{d}$$

or $$b > \sqrt{\frac{4000 \times 800}{100}} \; (= 178.88)$$

(ii) *From Moment of Resistance Consideration*

Loads

Self = 0.85 x 0.4 x 1 x 25	=	8.50 kN/m
Super imposed load	=	40.00 kN/m
Total load	=	48.30 kN/m
Maxm. B.M = $M = \frac{wl_{ef}^2}{2} = \frac{48.5 \times 4^2}{2}$	=	388.00 kNm

$$d = \sqrt{\frac{M}{R_B b}} = \sqrt{\frac{388 \times 10^6}{0.867 \times 400}} = 1057.73$$

$D = 1057.73 + 25 + 20 + 12.5 = 1115.23$

Taking $D = 1150$

$d = 1150 - 25 - 20 - 12.5 = 1092.5$

Checking for Lateral Stability

$$l_{ef} < \frac{100b^2}{d}$$

or $$b > \sqrt{\frac{4000 \times 1092.5}{100}} \quad (= 209)$$

Loads

Self = 1.15 × 0.4 × 1 × 25	=	11.50 kN/m
Super imposed load	=	40.00 kN/m
Total load	=	51.50 kN/m

$$M = \frac{w l_{ef}^2}{2} = \frac{51.5 \times 4^2}{2} = 412.00 \text{ kNm}$$

$$d = \sqrt{\frac{M}{R_b b}} = \sqrt{\frac{412 \times 10^6}{0.867 \times 400}} = 1089.5$$

Taking $D = 1150$

$d = 1150 - 25 - 20 - 12.5 = 1092.5$

$$A_{st} = \frac{M}{\sigma_{st} j_B d} = \frac{412 \times 10^6}{140 \times 0.867 \times 1092.5} = 3106.9 \text{ mm}^2$$

Hence provided 12ϕ20 in two layers (A_{st} = 3770 mm²)

Curtailment of Steel

Curtailing all the six ϕ20 bars of the lower layer (Figure 11.6)

$d = 1150 - 50 - 10 = 1115$

and taking moment of area about N.A.

$$\frac{b \times (kd)^2}{2} = mA_{st}(d - kd)$$

$$\frac{b \times k^2 d}{2} = mA_{st}(1 - k)$$

$$\frac{400k^2 \times 1115}{2} = 19 \times 6 \times 314 \times (1 - k)$$

$223000\, k^2 = 35796 - 35796\, k$

$223000\, k^2 + 35796\, k - 35796 = 0$

$$k = \frac{-35796 \pm \sqrt{3.321138562 \times 10^{10}}}{2 \times 223000}$$

$k = 0.328 < 0.404\ (k_B)$

Hence the section is under-reinforced.

$$\therefore \; M_R = A_{st} jd = 140 \times 6 \times 314 \times \left(1 - \frac{0.328}{3}\right) 1115 \times 10^{-6}$$

$$= 261.938 \text{ kNm}$$

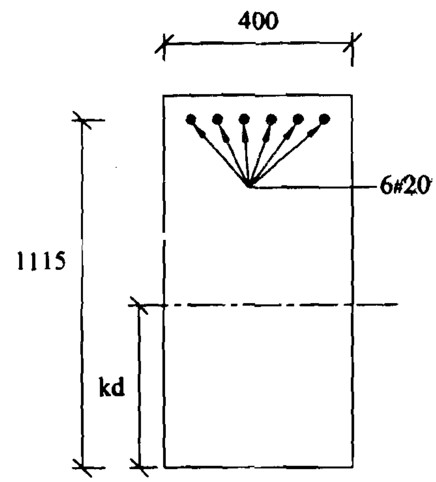

Figure 11.6 : Section of the Beam after Curtailment

Let x be the distance from free end where 6 lower bars can be curtailed (Figure 11.5)

$$\text{i.e. } 261.938 = w \times \frac{x^2}{2} = 51.5 \times \frac{x^2}{2}$$

or $\quad x = 3.189$ m

Actual cut-off point from *free end*

$= 3.189 - (12\phi \text{ or } d)$

$= 3.189 - 1.115 = 2.074 > 1.167\text{m} \; (L_d)$

i.e. distance of actual curtailment from *fixed end* = 4-2.074 = 1.925 > 1.167 (L_d).....(Figure 11.7)

Inside the support, length of the bar to be provided *= L_d = 1167 = 515 + 8ϕ + x

or $\quad$ x = 492 = length of straight bar to be extended beyond bend

Provision for Shear

(i) *At support*

Maximum Shear Force, $V = wl_{ef} = 51.5 \times 4 = 206$ kN

$$\tau_v = \frac{V}{bd} = \frac{206 \times 1000}{400 \times 1092.5} = 0.471 \text{ N/mm}^2$$

$$\frac{100 \times A_{st}}{bd} = \frac{100 \times 12 \times 314}{400 \times 1092.5} = 0.862 \text{ N/mm}^2$$

$$\tau_c = 0.34 + \frac{(0.862 - 0.75)}{0.25} \times (0.37 - 0.34) = 0.353$$

$$\therefore \; V_s = V - \tau_c bd = 206 - 0.353 \times 400 \times 1092.5 \times 10^{-3} = 51.739 \text{ kN}$$

* As the width of support = 600, straight portion of bar inside the column (Figure 11. 7)
= 600 - (clear cover + dia of bar bend + radius of bend)
= 600 – (25+ ϕ + 2ϕ) = 600 – (25 + 3 × 20) = 515

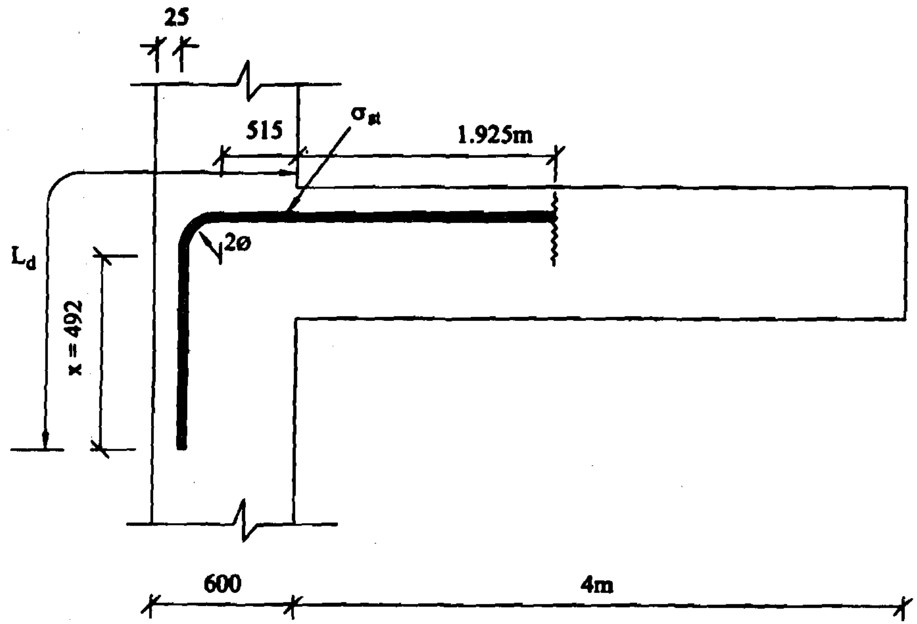

Figure 11.7 : Extension of Main Reinforcement in the Support for Development Length

Adopting ϕ8 two-legged verical stirrups

$$V_s = \frac{\sigma_{sv} \times A_{sv} \times d}{s_v}$$

$$s_v = \frac{\sigma_{sv} \times A_{sv} \times d}{V_s} = \frac{140 \times 100.53 \times 1092.5}{51.739 \times 10^3} = 297.185$$

From minimum shear force reinforcement consideration, maximum spacing of stirrups are given by

(a) $s_v = \frac{0.87 f_y \times A_{sv}}{0.4 b} = \frac{0.87 \times 250 \times 100.53}{0.4 \times 400} = 136.65$ c/c

(b) $s_v = 0.75\, d = 0.75 \times 1092.5 = 819.375$

(c) $s_v = 450$ mm

$\therefore$ **Provided ϕ8 two-legged vertical stirrups @ 135**

(ii) *At cut off point*

Total shear reinforcement

$$A_{sv} + A_{sve} = \frac{V_s \times s_v}{\sigma_{sv} d} + \frac{0.4 b s_v}{f_y} = s_v \left(\frac{V_s}{\sigma_{sv} d} + \frac{0.4 b}{f_y} \right)$$

(Adopting ϕ8 two-legged verical stirrups, $A_{sv} + A_{sve} = 100.53$ mm^2)

or $$100.53 = s_v \left(\frac{10^3}{140 \times 1115} + \frac{0.4 \times 400}{250} \right)$$

$s_v = 103.48$

Hence provided ϕ8 two-legged vertical stirrups @ 100 for a distance of

$\frac{3}{4} \times 1115 = 836.25(=900)$ **from actual cut-off point section**

As $D > 750$, side reinforcement reqd., $A_s = 0.1$ % of Total Web Area

$= \frac{0.1}{100} \times 400 \times 1150 = 460$ mm² i.e. $460/2 = 230$ mm² on each face at spacing not exceeding 300.

The detailing of reinforcement has been shown in Figure 11.8.

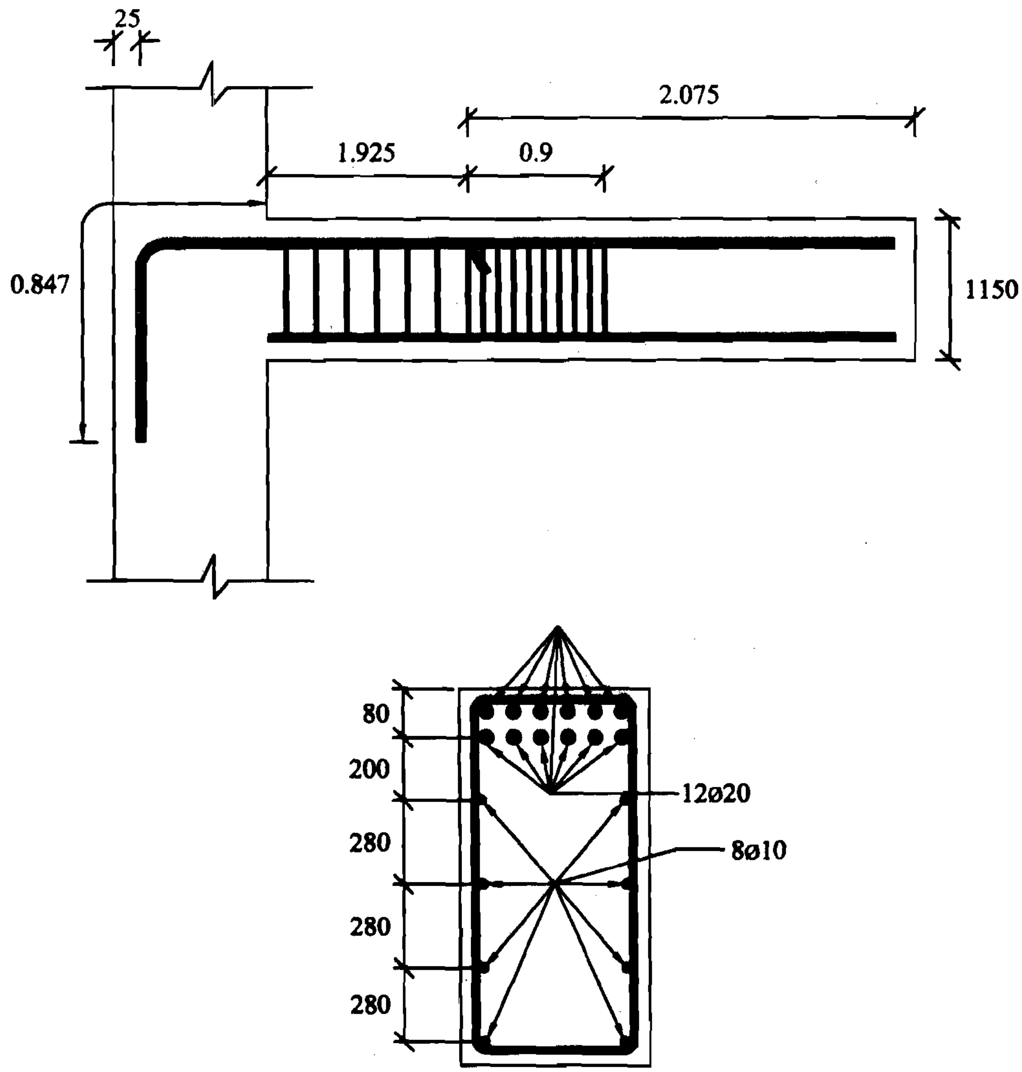

Figure 11.8 : Detailing of Reinforcement

Example 11.3

Design a continuous beam for the following data Figure 11.9 :

Super imposed dead load= 10 kNm

Super imposed live load = 12 kNm

Width of support = 350 mm

Use M 15 concrete and Fe 250

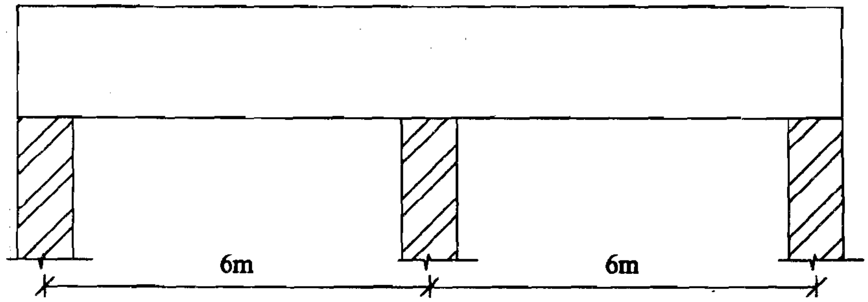

Figure 11.9 : A Continuous Beam

Solution

Depth (*D*)

(i) *From Thumb Rule*

$\frac{l_{ef}}{d}$ lying between 10 to 20

Let $\frac{l_{ef}}{d} = 10$

or $d = \frac{6000}{10} \approx 600$ mm

(ii) *From deflection criteria*

$$\frac{l_{ef}}{K_1 K_2 K_3 d} = \frac{(20+26)}{2d} = 23 = {*}K_B$$

$$p_B\% = \frac{k_B \sigma_{cbc}}{2\sigma_{st}} \times 100 = \frac{0.404 \times 5}{2 \times 140} \times 100 = 0.714$$

Corresponding to $p_B\% = 0.714$, $K_1 = 1.68$

$K_2 = K_3 = 1$

$$\therefore d = \frac{6000}{23 \times 1.68} = 155.28$$

(iii) *From Moment of Resistance Consideration*

Taking $D = 760$ mm

Loads

Self load = 0.76 × 0.3 × 25	=	5.70 kN/m
DL	=	10.00 kN/m
Total *DL*	=	15.70 kN/m
LL	=	12.00 kN/m
Total Load (*DL* + *LL*)	=	27.70 kN/m

* Here K_B is the average basic value for $\frac{l_{ef}}{d}$ for simple and continuous supports

Evaluation of BMs and SFs at Critical Sections (Figure 11.10)

(vide Table A1 & A2 of Appenidx A of Unit 6)

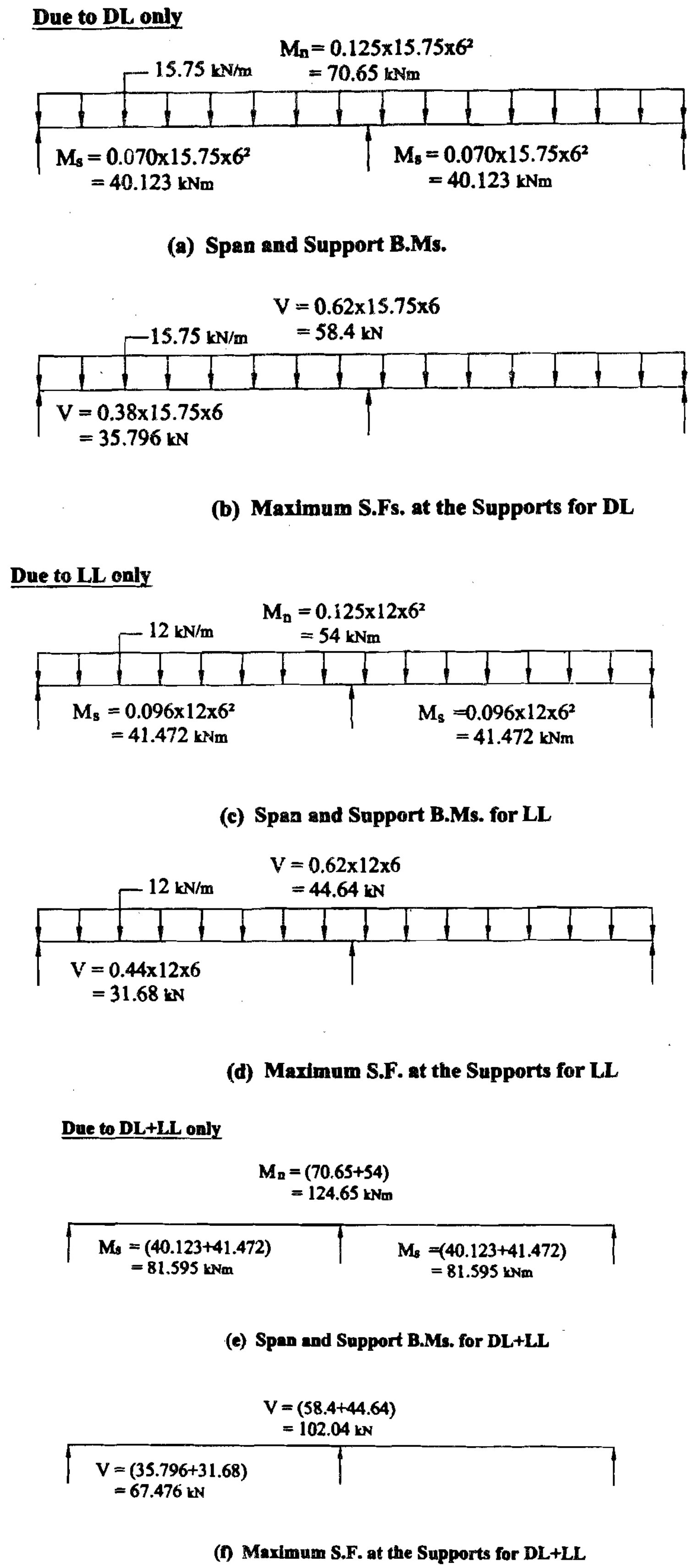

Figure 11.10 : Evaluation of B. Ms and S. Fs at Critical Sections

Maximum B.M.(M) occurs at intermediate support,

$$d = \sqrt{\frac{M}{R_B d}} = \sqrt{\frac{124.65 \times 10^6}{0.867 \times 300}} = 692.27$$

$D = 692.27 + 25 + 10 = 727.27$

Provided D = 760; d = 760 – 25 – 10 = 725

$$A_{st} = \frac{M}{\sigma_{st} j_B d} = \frac{124.65 \times 10^6}{140 \times 0.867 \times 725} = 1416.47 \text{ mm}^2$$

Hence provided 5ϕ20 (A_{st} = 1570 mm²) at intermediate support

Moment of Resistance of the Section for Intermediate Support Section (Figure 11.11).

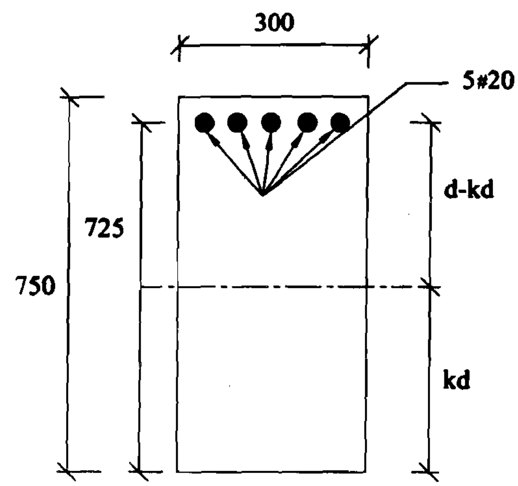

Figure 11.11 : Section at Support

Taking moment of area about N.A.

$$b \times kd \times \frac{kd}{2} = mA_{st}(d - kd)$$

$$\frac{b}{2} \times k^2 d = mA_{st}(1 - k)$$

$$\frac{300}{2} \times k^2 \times 725 = 19 \times 314 \times 5\,(1 - k)$$

$$108750\,k^2 + 29830\,k - 29830 = 0$$

$$k = 0.404 = k_B\ (=0.404)$$

$$\therefore\ M_R = \sigma_{st} A_{st} jd$$

$$= 140 \times 5 \times 314 \times 0.867 \times 725$$

$$= 138.16 \text{ kNm}$$

For +ve B.M

$$A_{st} = \frac{M}{\sigma_{st} j_B d} = \frac{81.595 \times 10^6}{140 \times 0.867 \times 725} = 927.21 \text{ mm}^2$$

Hence provided 5ϕ16 as positive reinforcement at mid span

Moment of resistance of the section at mid span

Taking moment of area about N.A. (Figure 11.12).

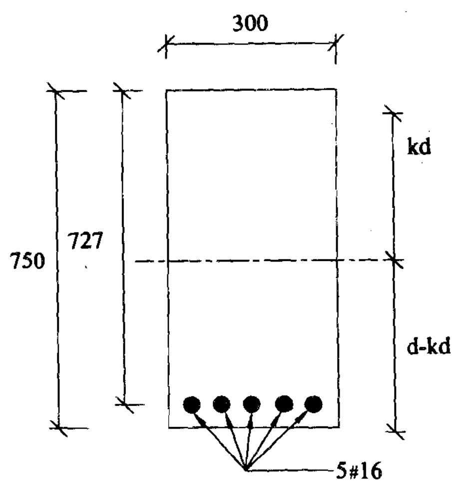

Figure 11.12 : Section at Mid Span

$$b \times kd \times \frac{kd}{2} = mA_{st}(d - kd)$$

$$\frac{b}{2} \times k^2 d = mA_{st}(1 - k)$$

$$\frac{300}{2} \times k^2 \times 727 = 19 \times 1005.3 \times 5 (1 - k)$$

$$109050\, k^2 + 19100.7\, k - 19100.7 = 0$$

$k = 0.34$

$k < k_B$ (=0.404)

The section is under-reinforced

$$\therefore\ M = \sigma_{st} A_{st} jd$$

$$= 140 \times 1005.3 \left(1 - \frac{0.34}{3}\right) \times 727$$

= 90.72 kNm>

The detailing of the reinforcement has been shown in the Figure 11.13.

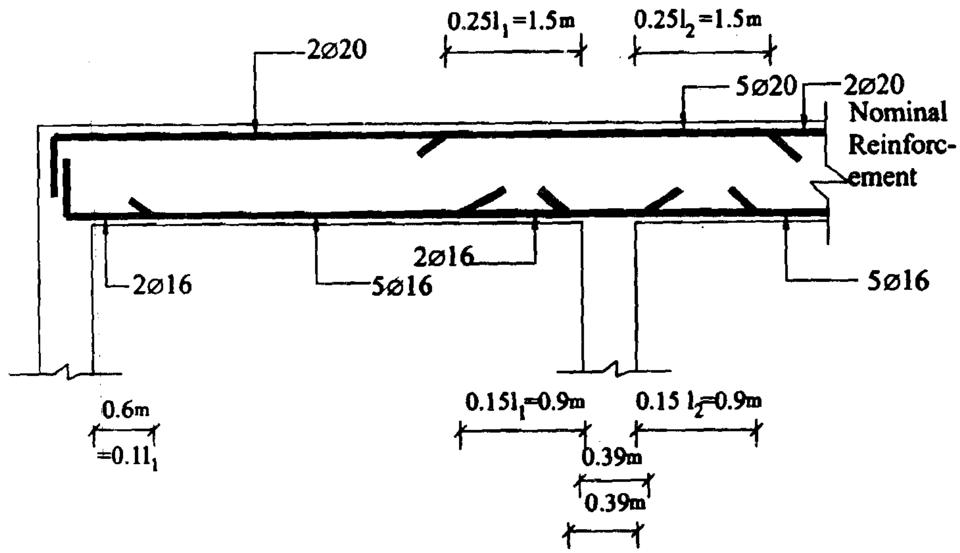

Figure 11.13 : Reinforcement Detailing for the Designed Beam

11.3 DESIGN OF LINTEL

11.3.1 General

Lintel is a beam provided over an opening to support loads from masonry (allowing for arching and dispersion, where applicable) and from any other part of the structure.*

The *design* of a lintel is done exactly in the same way as that of a rectangular beam except that the minimum shear reinforcement requirements as per code may not be complied with where the maximum shear stress calculated is less than half the permissible value.

Length of bearing of lintel on each side shall neither be less *90 mm* nor less than $\frac{1}{10}$ th of the span.

Area of bearing on each side should be sufficient to transfer the reaction of lintel and laod due to arching action without exceeding the permissible stresses on masonry.

11.3.2 Evaluation of Design Loads on a Lintel

The selfweight of a wall or any other load from structural component upon it *over an opening* are transferred to the sides of the opening through a lintel and by arching action. The proportion of the above mentioned loads carried by lintel depends upon support condition, height of masonary above the opening, location of concentrated or distributed gravity loads etc.

For *arching action* to take place, masonary must have good shearing strength, good bond and enough spread of wall on both sides of an opening.

Arching action transfers the loads of floor and masonary above the *equilateral* triangle to the sides of wall. Any other opening or load below the horizontal plane, 250 above the apex of the Δ^{bc}, effects the loading on the lintel (Figure 11.14)

* Arching is a phenomenon by which part of the load over an opening in the wall gets transferred to the sides of the opening.

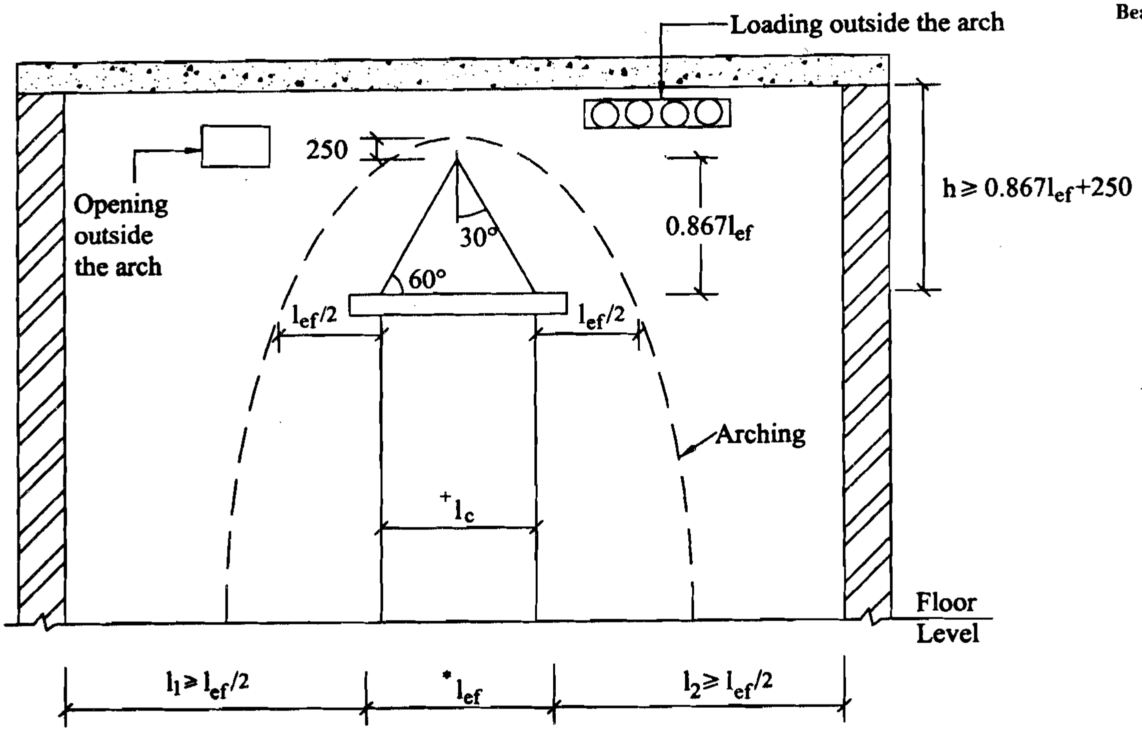

Figure 11.14 : Diagrammatic Representation of Arching Action and Design Load when l_1 & $l_2 \geq l_{ef}/2$ and $h \geq 0.867\, l_{ef} + 250$

Keeping above facts in view design loads on a lintel is evaluated as under :

Condition I *When wall above the opening, $h \geq (0.867\, l_{ef} + 250)$ high, and supporting walls are spread for more than $\frac{l_{ef}}{2}$ on both sides of the opening* Figure 11.14

Due to arching action shown by thick dotted line Figure 11.14, the load of masonry on the lintel (W) is only that of an *Equilateral Triangular Portion* of wall formed on effective span (l_{ef}) as base regardless of any other opening or load outside the dotted arch.

In other words,

Design Load, W = Weight of Equilateral Δ^{ar} portion of masonary wall on lintel.

$$= \frac{1}{2} l_{ef} \times 0.867 l_{ef} \times t \times \rho \qquad \ldots (11.1)$$

where t = thickness of wall and ρ = density of masonry

$$\text{Maximum B.M., } M_{max} = \frac{1}{2} W \frac{l_{ef}}{2} - \frac{W}{2} \cdot \frac{l_{ef}}{2} \cdot \frac{1}{3} = \frac{W l_{ef}}{6} \qquad \ldots (11.2)$$

$$\text{and maximum S.F., } V_{max} = \frac{W}{2} \qquad \ldots (11.3)$$

\+ l_c = clear span of the opening

* l_{ef} is the same as that for simply supported beam given in the Code.

Condition II *When l_1 and /or $l_2 < \frac{l_{ef}}{2}$*

In case if wall on any one or both sides of the opening spreads less than half the effective span of lintel, the design load is equal to the sum of the masonry and floor load falling above the effective span of the lintel– regardless of the height of the floor slab (Figure 11.15)

In other words,

$$W = l_{ef} \times h \times \rho \times r + l_{ef} \times \text{Floor Load/Unit length}$$

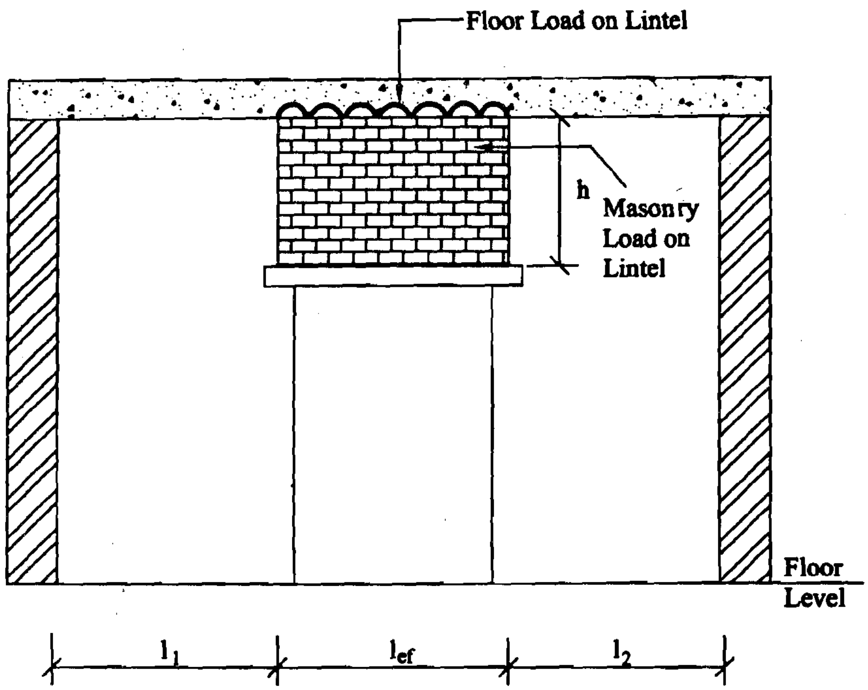

Figure 11.15 : Design Load on Lintel when l_1 and / or $l_2 < l_{ef} / 2$

Condition III *When l_1 & $l_2 \geq \frac{l_{ef}}{2}$ and floor or roof slab falls within a part of the triangle*

In this case design load consists of trapezoidal portion (ABCD) of the triagnular load below the floor slab and floor load falling within the triangle. If the wall continuous on the upper floor or there is a parepet wall over the wall, the triangular portion (EFG) of the wall will be included in the design load (Figure 11.16).

In other words,

Design load, $W =$ Wt. of masonry wall (ABCD) (or wt. of masonary wall ABCD + wt. of masonary wall EFG if the wall continuous above floor) + length CD × wt. of floor/unit length

Condition IV *When l_1 & $l_2 \geq \frac{l_{ef}}{2}$ and $h < 0.867\, l_{ef}$ and another opening comes within the horizonal plane 250 above the Δ^{lc}* (Figure 11.17)

The lintel of floor A may be designed for

Design laod W_A = Masonry Load of Storey A on lintel (Area ABGH)

+

Load of floor A on CD

+

Load of floor B plus masonry load of storey B on lintel

+

Masonry Load on lintel of floor B on l^B_{ef}

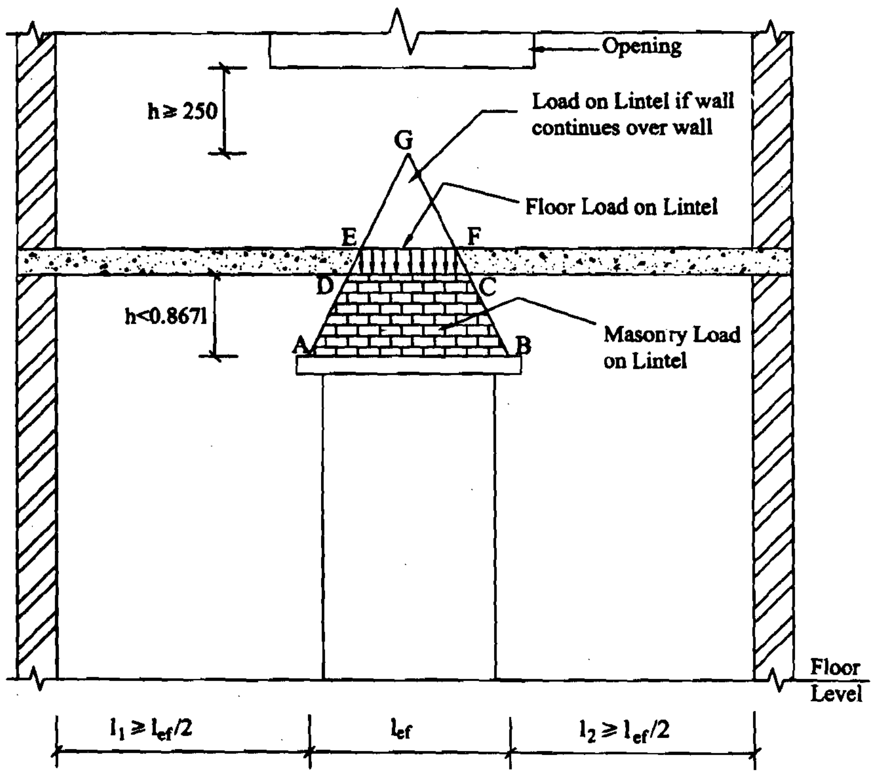

Figure 11.16 : Design Load when l_1 & $l_2 \geq l_{ef}/2$ and $h < 0.867\ l_{ef}$

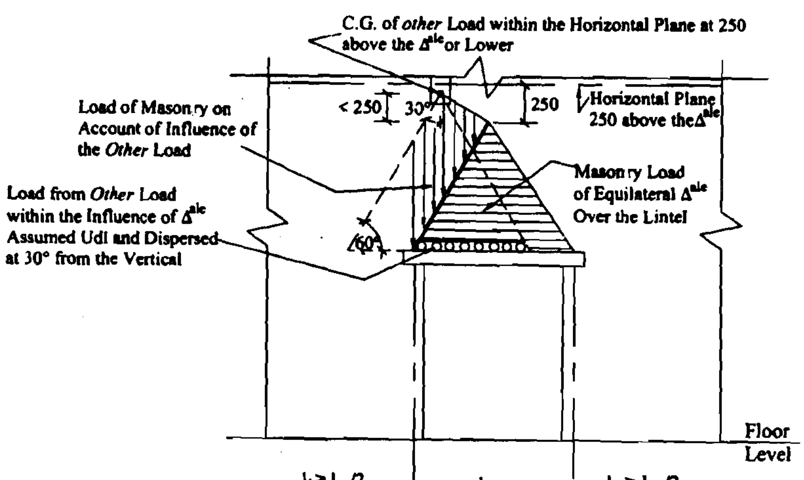

Figure 11.17 : Load when l_1 & $l_2 \geq l_{ef}/2$ and the Δ^{ale} on Lintel is within the Influence of any other Load

Condition V *When l_1 & $l_2 \geq \frac{l_{ef}}{2}$ and any other load lying between the lintel and horizontal plane 250 above the triangle* (Figure 11.18)

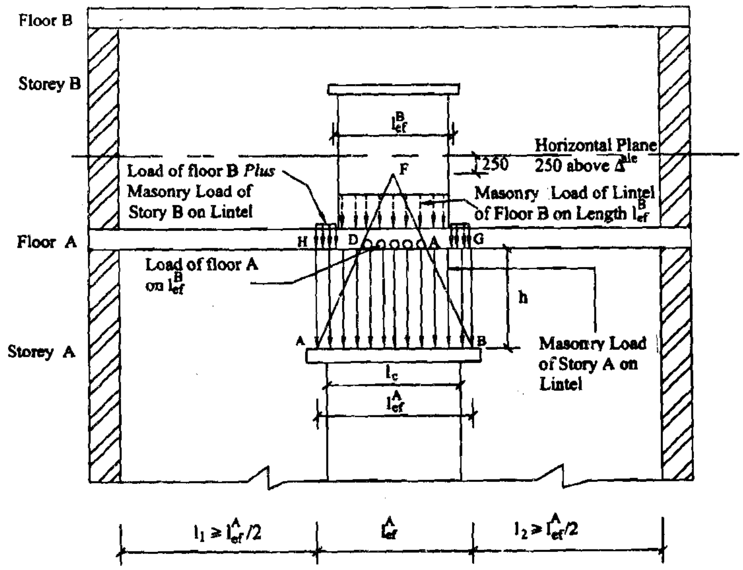

Figure 11.18 : Load on Lintel when l_1 & $l_2 \geq l^A_{ef}/2$ and another opening on Upper Floor falling within Horizontal Plane 250 above the Equilateral

Design Load, W = Masonry Load of equilateral triangle over the lintel

\+

Masonry load on account of influence of the *other* load

\+

Load from the *other* load within the influence of equilateral triangle assuming *udl* and dispersed at an angle of 30^0 from the vertical

Example 11.4

Design a lintel for 2m wide central opening in a brick wall of total length of 3.5m of a room of total inside dimension 3.5m × 3m for the following data :

(i) thickness of wall = 40

(ii) thickness of R.C. roof slab = 120

(iii) Lime terrace thickness = 150

(iv) parapet wall over the roof of 1m height and 100 thick

(v) Height of wall above lintel = 1.00 m

Use M 15 concrete and Fe 250 steel

Solution

As the spread of the wall on both sides of the opening are less than $\frac{l_{ef}}{2}$ and height of wall above lintel is less than 0.867 l_{ef}, the load on the lintel will be that as shown in *condition II.* Assuming total thickness of lintel 270 and effective cover 35 and bearing on each side = 250.

$l_{ef} = 2 + 0.235 = 2.235$ m

or

$l_{ef} = 2 + 0.25 = 2.25$ m

$\therefore\ l_{ef} = 2.25$ m

Loads

self weight of lintel	$= 2.25 \times 0.4 \times 0.27 \times 25$	=	6.075 kN
wt. of wall above lintel	$= 2.25 \times 0.4 \times 1 \times 18.85$	=	16.965 kN
wt. of slab	$= 2.25 \times 1.5 \times 0.12 \times 25$	=	10.125 kN
wt. of lime concrete	$= 2.25 \times 1.5 \times 0.15 \times 19$	=	9.619 kN
live load	$= 2.25 \times 1.5 \times 1.5$	=	5.063 kN
	Total Load W	=	47.847 kN
$\therefore$ Maximum B.M.	$= \frac{Wl_{ef}}{8} = \frac{47.847 \times 2.25}{8}$	=	13.457 kNm

Design coefficients

$$m = \frac{280}{3\sigma_{cbc}} = \frac{280}{3 \times 5} \approx 19$$

For balanced section

$$k_B = \frac{m\sigma_{cbc}}{m\sigma_{cbc} + \sigma_{st}} = \frac{19 \times 5}{19 \times 5 + 140} = 0.404$$

$$j_B = \left(1 - \frac{k_B}{3}\right) = \left(1 - \frac{0.404}{3}\right) = 0.865$$

$$M_{RB} = \frac{1}{2}\sigma_{cbc} b k_B d\left(d - \frac{k_B}{3}\right)$$

$$= \frac{1}{2}\sigma_{cbc} k_B j_B bd^2$$

$$= \frac{1}{2} \times 5 \times 0.404 \times 0.865\ bd^2$$

$$= 0.874\ bd^2$$

or $$d = \sqrt{\frac{13.457 \times 10^6}{0.874 \times 400}} = 196.2$$

$\therefore\ D = 196.2 + 25 + 6 = 227.2 \approx 270$ (adopted)

$\therefore\ d = 270 - 25 - 6 = 239$ (using ϕ12 bars)

A_{st}

$$A_{st} = \frac{M}{\sigma_{st} j_B d} = \frac{13.457 \times 10^6}{140 \times 0.874 \times 239} = 460.16 \text{ mm}^2$$

Hence provided 5ϕ12 bars (A_{st} = 565.49 mm² > 460.16 mm²)

$$\frac{A_{st,min}}{bd} = \frac{0.85}{f_y} = \frac{0.85}{250}$$

or $$A_{st,min} = \frac{0.85}{250} \times 400 \times 239 = 325.04 < 565.49 \text{ mm}^2)$$

Bent up 2ϕ12 at $\frac{l_{ef}}{7} = \frac{2.25}{7} \approx 300$ from inner edge of support

Provision for shear

S.F. at d from inner edge of support

$$V = \frac{W}{2} = \frac{47.847 \times 2}{2.25 \times 2} = 21.265 \text{ kN}$$

$$\therefore \tau_v = \frac{V}{bd} = \frac{21.265 \times 10^3}{400 \times 239} = 0.222 \text{ N/mm}^2$$

At support, $A_{st} = 3 \times \frac{\pi}{4} \times 12^2 = 339\text{mm}^2$

$$\therefore \frac{100 A_{st}}{bd} = \frac{100 \times 339}{400 \times 239} = 0.35$$

From Table 8.3

$$\tau_c = 0.22 + \frac{(0.29 - 0.22) \times (0.355 - 0.25)}{0.25} = 0.249 \text{ N/mm}^2 > 0.222 \text{ N/mm}^2$$

Since $\tau_v < \tau_c$, but $\tau_v > \frac{\tau_c}{2}$ only nominal reinforcement will be provided.

Adopting £6 two-legged vertical stirrups

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

or $$s_v \leq \frac{0.87 f_y A_{sv}}{0.4b} = \frac{0.87 \times 250 \times 56}{0.4 \times 400} = 76.125$$

The spacing of stirrups shall not also exceed the following

(i) 0.75 d = 0.5 × 233 = 179.25

(ii) 450

Hence provided ϕ6 two-legged vetical stirrups @ 75 througout

Check for Development Length of Support

$$1.3 \frac{M_1}{V} + L_0 \geq L_d$$

with 3ϕ12 at support the section is under-reinforced, and therefore first of all n.a. depth be calculated to know M_1

Taking moment of equivalent concrete areas about n.a.

$$b\frac{x^2}{2} = mA_{st}(d-x)$$

or $$400\frac{x^2}{2} = 19 \times 339(339-x)$$

or $$220x^2 + 6441\,x - 159399$$

or $$x^2 + 32.21\,x - 7697 = 0$$

or $$x = \frac{-32.21 \pm \sqrt{32.21^2 \times 30788}}{2} = 73.093$$

$$x_B = 0.4\,04 \times 239 = 96.556 > 73.093$$

$$M_1 = \sigma_{st} A_{st} jd = 140 \times 339 \times \left(239 - \frac{73.093}{3}\right) = 10.187 \text{ kNm}$$

$$V = \frac{47.857}{2} = 23.93 \text{ kN}$$

or $$L_0 = \frac{L_s}{2} - x' + 13\phi \text{ for standard hook}$$

where L_s = support width = 250

x' = side cover = 25

or $$L_0 = \frac{250}{2} - 25 + 13 \times 12 = 256$$

$$L_d = \frac{\sigma_s \phi}{4\tau_{bd}} = \frac{140\phi}{4 \times 0.6} = 58.3\phi = 58.3\phi \times 12 = 699.6$$

$$1.3\frac{M_1}{V} + L_0 = 1.3\frac{10.187 \times 10^6}{23.93 \times 10^3} + 256 = \mathbf{809.4\ mm > 699.6\ Hence\ O.K.}$$

SAQ 1

(i) Design and detail a simply supported R.C. beam of clear span 7m supported on 400 mm wall on both sides. The beam is loaded with a dead load of 20 kN/m and a live load of 12 kN/m. Use M 20 concrete and Fe 415 steel.

(ii) Design and draw a R.C. cantilever beam of 3.5 m span loaded with 45 kN/m of design laod inclusive of self load. The beam is projecting from a R.C. column. Use M 20 concrete and Fe 415 steel.

(iii) Design and detail a lintel for the specifications of Example 11.4 except that the wall horizontal length is 5m instead of 3.5m.

11.4 SUMMARY

Design of three types of recatangular R.C. beams have been done for design (actual or characteristic) loads keeping in view their safety by ensuring that permissible stresses (allowable stresses in working stress design) are no where exceeded. Secondly, while fixing the size of concrete and steel and detailing the reinforcements, control of deflection and cracking (serviceability limit) were the main concern. The design of a lintel is the same as that of a simply supported beam except that the loading over it is different due to arching action. Variation in loading patterns on lintel due to different parameters and evaluation of corresponding design load have been explained.

11.5 ANSWERS TO SAQs

SAQ 1

(i) Refer Example 11.1

(ii) Refer Example 11.2

(iii) Refer Example 11.4 using condition III of 11.3.2