UNIT 5 BOND AND ANCHORAGE

Structure

5.1 INTRODUCTION

In previous Units 2, 3 & 4, analysis and design of members for flexure, shear and torsion were discussed. Concrete as well as reinforcements were provided in adequate quantities and at proper locations to resist tensile as well compressive stresses due to above mentioned applied forces. Reinforced concrete being a composite material has to have the same compatible strains and deformations at a point of a section both in concrete as well as in reinforcements. This is possible only when there is proper bond between them. Bond keeps both concrete and steel in position and prevent slippage relative to each other when stressed. For example, the same total deformation, Δ, both in concrete and in reinforcements in a column (Figure 5.1(a)) had been possible because of proper bond between them (If there were no proper bond or any bond between concrete and steel, concrete would have deformed more than reinforcements). Similarly, elongation of longitudinal fibres of concrete around the reinforcing bars would have been more than those of bars at the same location, had there been no bond between them (Figure 5.1(b)).

Bond Stress, therefore, can be defined as *the longitudinal shear stress at the interface between concrete and reinforcements.*

Bond is developed due to

i) **adhesion** of laitance* at the interface,

ii) **friction** between the two materials of the interface after the adhesion fails at very low stress, and

iii) **mechanical resistance** due to twisted bars or end anchorage after failure of bond due to adhesion.

Special provisions are made for development of proper bond

a) where there is large variation of bending moment over a short distance (i.e. high shear force), and

b) also where some of the reinforcing bars are terminated (curtailed).

* gel formed of cement and water is called laitance.

Sometimes splicing (joint between two bars with proper bond) is done where a bar in one continuous length cannot be provided.

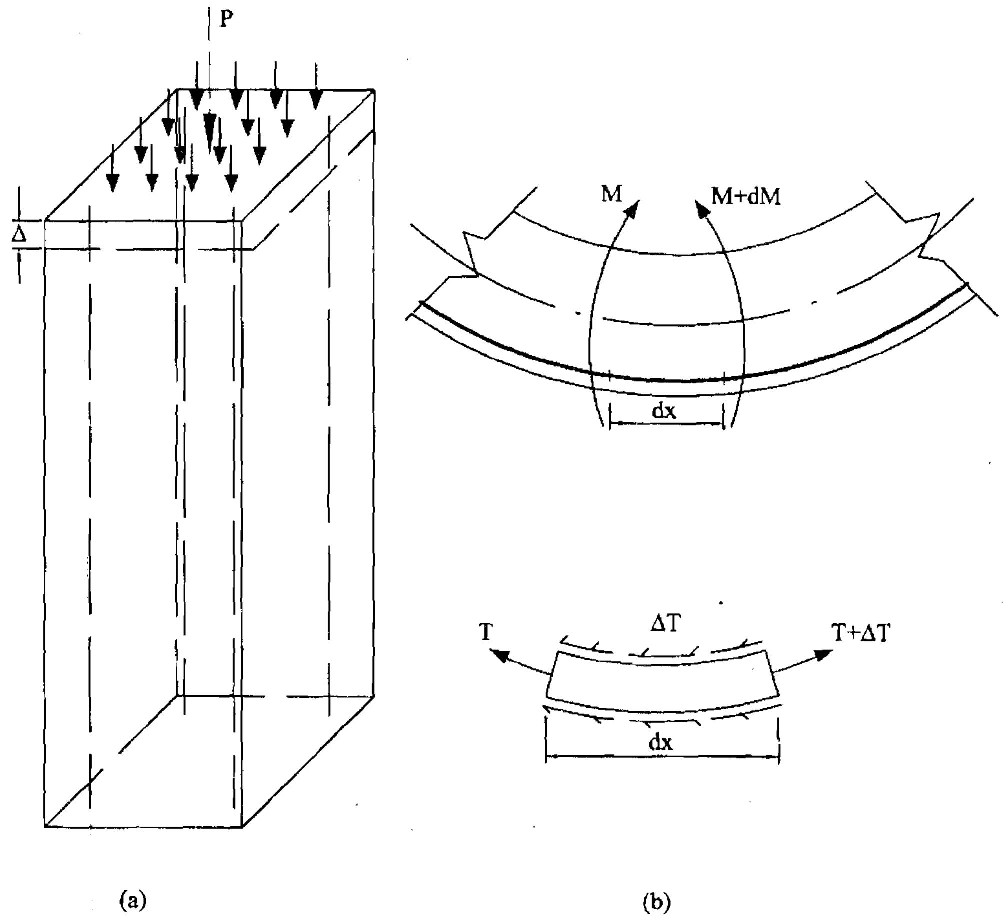


Figure 5.1 : Explaining the Effect of Proper Bond between Concrete and Reinforcement

SAQ 1

Define bond stress and discuss the mechanism of bond between concrete and reinforcements.

Objectives

After going through this unit students will learn about the following :

i) bond between concrete and reinforcing bars,

ii) types of bond,

iii) end anchorages of reinforcing bars and development length,

iv) curtailment of tension reinforcement in flexural members, and

v) splicing.

5.2 TYPES OF BOND

The bond developed along the length of a reinforcing bar of a flexural member is termed as **flexural bond**; whereas bonds at its ends and at cut-off points are known as **anchorage bond**.

5.2.1 Flexural Bond

Let dx be a piece of a reinforcing bar at a distance x from the free end of a cantilever shown in Figure 5.2. If f_b be the bond stress at the interface of concrete and reinforcement, then from equilibrium of forces,

$$-T - \pi\phi dx f_b + T + \Delta T = 0$$

$$\text{or, } f_b = \frac{\Delta T}{\pi\phi dx} = \frac{A_{st}\sigma_s}{\pi\phi dx} = \frac{\Delta M}{\sigma_s jd\pi\phi}\frac{\sigma_s}{dx} = \frac{dM}{dx} \times \frac{1}{jd\pi\phi}$$

where σ_s = stress in bar of diameter ϕ at the section at design load.

If there are n numbers of tensile reinforcement, then $\pi\phi$ may be replaced by $n\pi\phi$ or $\Sigma 0$ (where $\Sigma 0$ is the summation of perimeter of the bars). The above expression may be written as

$$f_b = \frac{V}{jd\Sigma 0} \qquad \ldots (5.1)$$

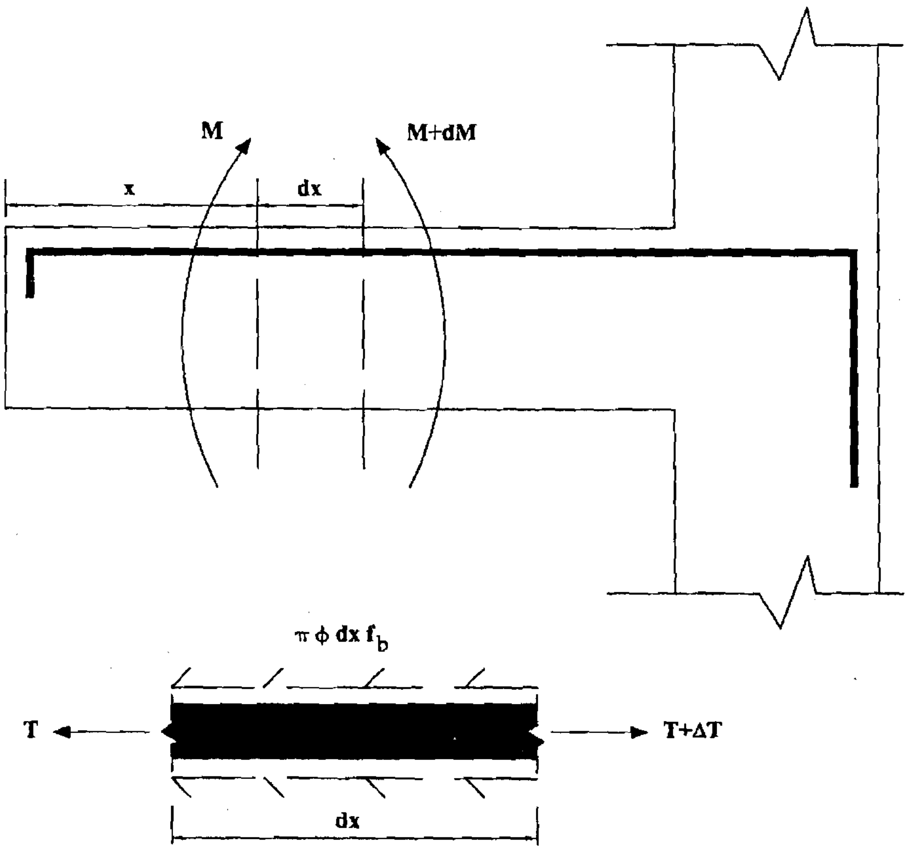


Figure 5.2 : Flexural Bond

However, this value of bond stress is not tallying with the actual one because of the following two reasons:

i) The formation of cracks at *discrete* intervals (Figure 5.3(a)) along longitudinal axis of a reinforcing bar causes large variations in *tensile strength* from local maximum at the cracks to the local minimum at the middle of uncracked regions, and

ii) The variation of *bond stress* along the length of a reinforcing bar is irrational (Figure 5.3(b)). The magnitudes of the bond stress do not tally with the calculated ones. For example, if for round bars, the variation is about 10% for pure flexure and about 30% for flexure combined with shear, it is even more for deformed bars.

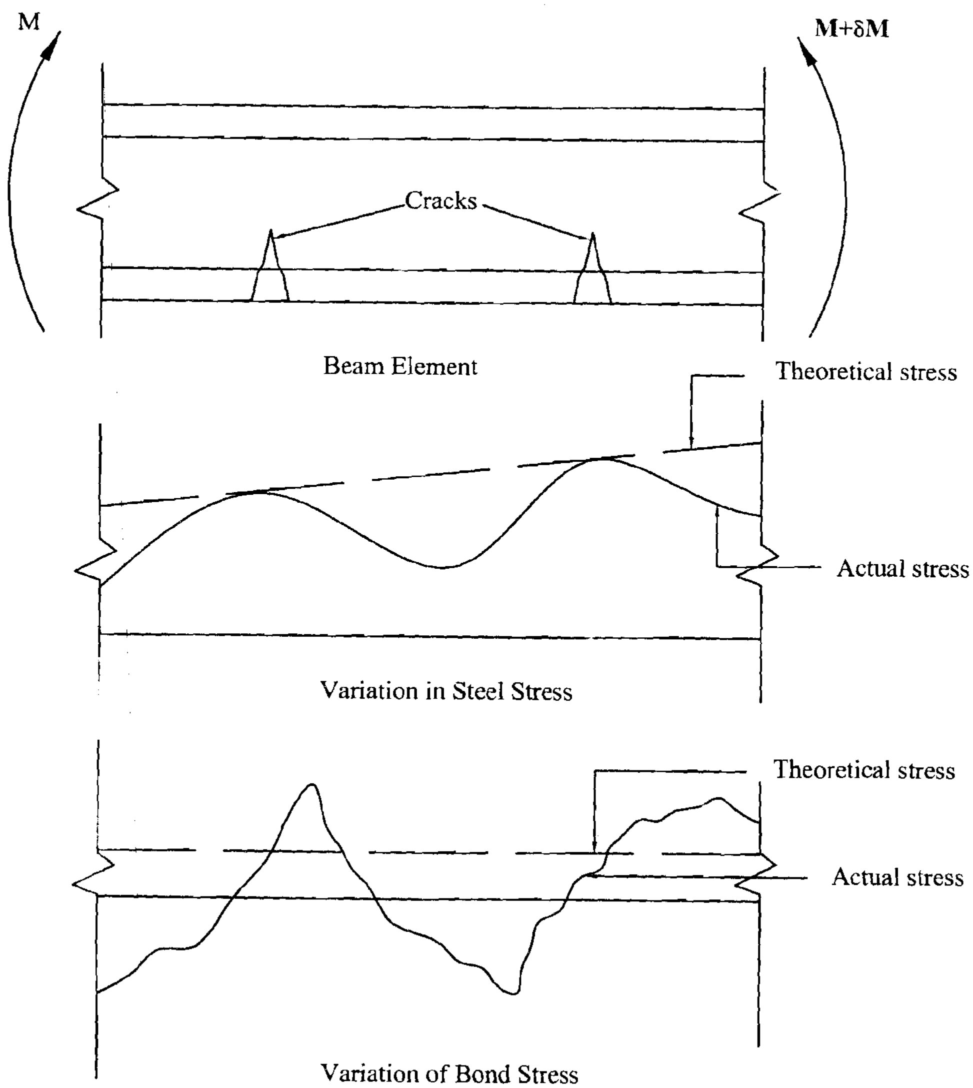


Figure 5.3 : Showing Variation between Actual Bond Stress and Theoretical Bond Stress

5.2.2 Anchorage Bond and Development Length

The reinforcing bar at ends or at cut-off section may slip at the interface if requisite length on each side of a section considered is not provided to develop the strains & stress at that section. Such length of a bar on each side of a section is termed as **Development Length (L_d)**. In practice, it is that length over which a pre-assigned slip will occur at design load for a *uniform* bond resistance. Assuming the *design* bond stress, T_{bd}, to be uniform on both sides of a section x-x (Figure 5.4)

$$T = \tau_{bd}\ \pi\ \phi\ L_d = \frac{\pi}{4}\phi^2\,\sigma_s$$

or,

$$L_d = \frac{\sigma_s\,\phi}{4\,\tau_{bd}} \qquad \ldots (5.2)$$

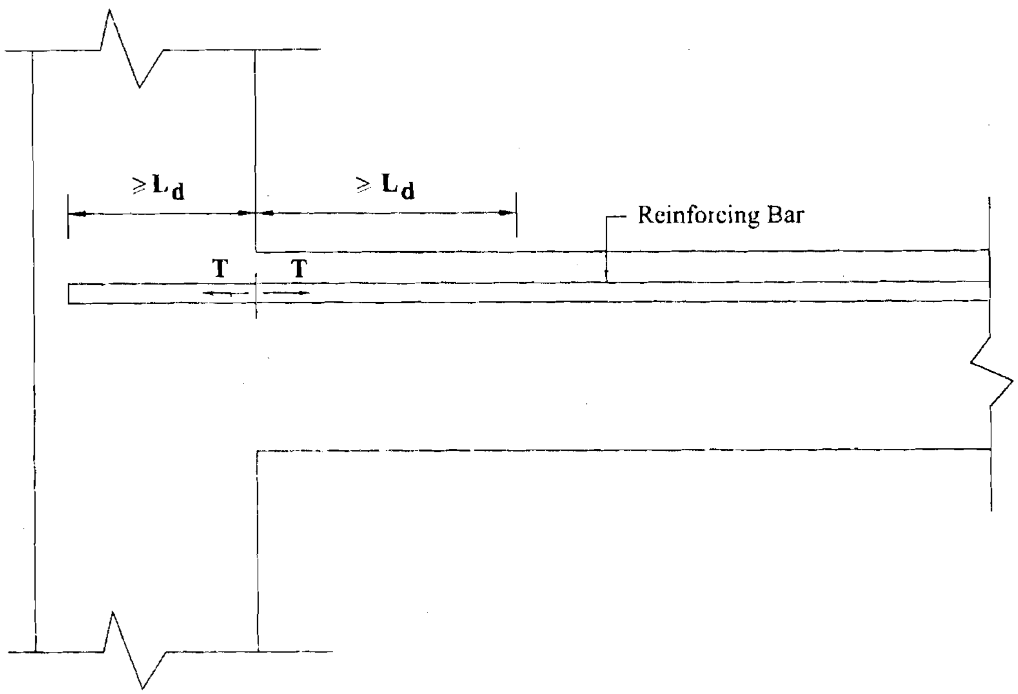


Figure 5.4 ; Evaluation of Development Length L_d

Design Bond Stress for plane bars in tension for different grade of steel shall be as given in Table 5.1

Table 5. 1: Design Bond Stress for Plain Bars in Tension τ_{bd}

Grade of Concrete	M 15	M 20	M 25	M 30	M 35	M 40
Design Bond Stress, τ_{bd} (N / mm²)	1.0	1.2	1.4	1.5	1.7	1.9

Note:

i) For **deformed bars** conforming to IS:1786-1979 or IS:1139-1966, the above values shall be increased by 60%.

ii) For **bars in compression**, the tabulated values of T_{bd} shall be increased by 25%.

Development length for different grades of concrete and steel calculated according to Eq. (5.2) are given in Tables 5.2 & 5.3.

Table 5.2 : Development Length (L_d) for Fully Stressed ($0.87f_y$) Single Bar in *Tension*

Steel grades	**Concrete grade**			
	M 15	M 20	M 25	M 30
Fe 250	55 ϕ	46 ϕ	39 ϕ	37 ϕ
Fe 415	57 ϕ	47 ϕ	41 ϕ	38 ϕ
Fe 500	68 ϕ	57 ϕ	49 ϕ	46 ϕ

Table 5.3 : Development Length (L_d) for Fully Stressed ($0.87f_y$) Single Bar in Compression

Steel grades	Concrete grade			
	M 15	M 20	M 25	M 30
Fe 250	44 ϕ	37 ϕ	32 ϕ	29 ϕ
Fe 415	46 ϕ	38 ϕ	33 ϕ	31 ϕ
Fe 500	55 ϕ	46 ϕ	39 ϕ	37 ϕ

The concept of development length calculated from anchorage bond gives better estimate of strength, and directly calculate the length of bar required on either side of a section to develop *design* stress.

However, *at simple support or at point of inflexion,* where there is large variation of bending moment over a short distance* (i.e. at high shear force) the development length criteria are to be complied both from flexural bond and anchorage bond considerations by equating τ_{bd} with f_{bd} for *fully stressed* bars.

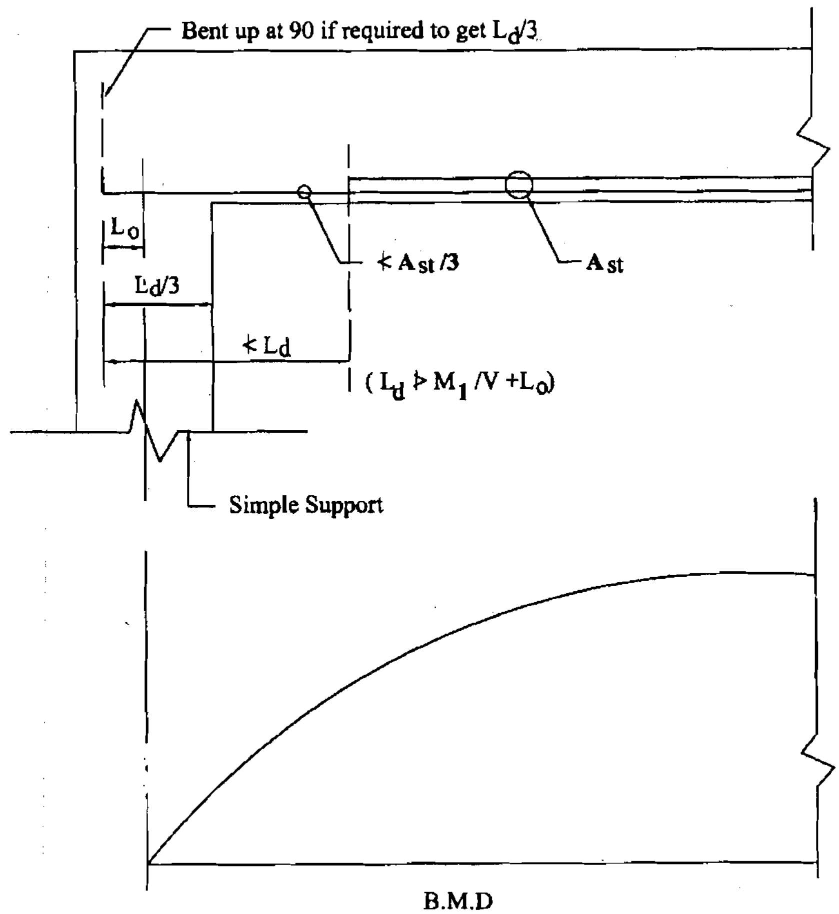


Figure 5.5 : Tensile Development Length at Simple Support for Beams and Slabs

Now, $f_{bd} = \tau_{bd}$

or, $\frac{\bullet\ V}{j\,d\sum 0} = \frac{f_d\,\phi}{4\,L_d}$ vide Eqs. 5.1 and 5.2

or, $L_d = \frac{f_d\,\phi\,j\,d\sum 0}{4\,V} = \frac{f_d\,\phi\,j\,d\,n\,\pi\,\phi}{4\,V}$

* a case of flexural load.

where M_1 = moment of resistance of the section assuming all reinforcement at the section to be stressed to f_d,

f_d = 0.87 f_y (vide Cl 25.2.3.3 of code), and

V = shear force at the section due to design loads.

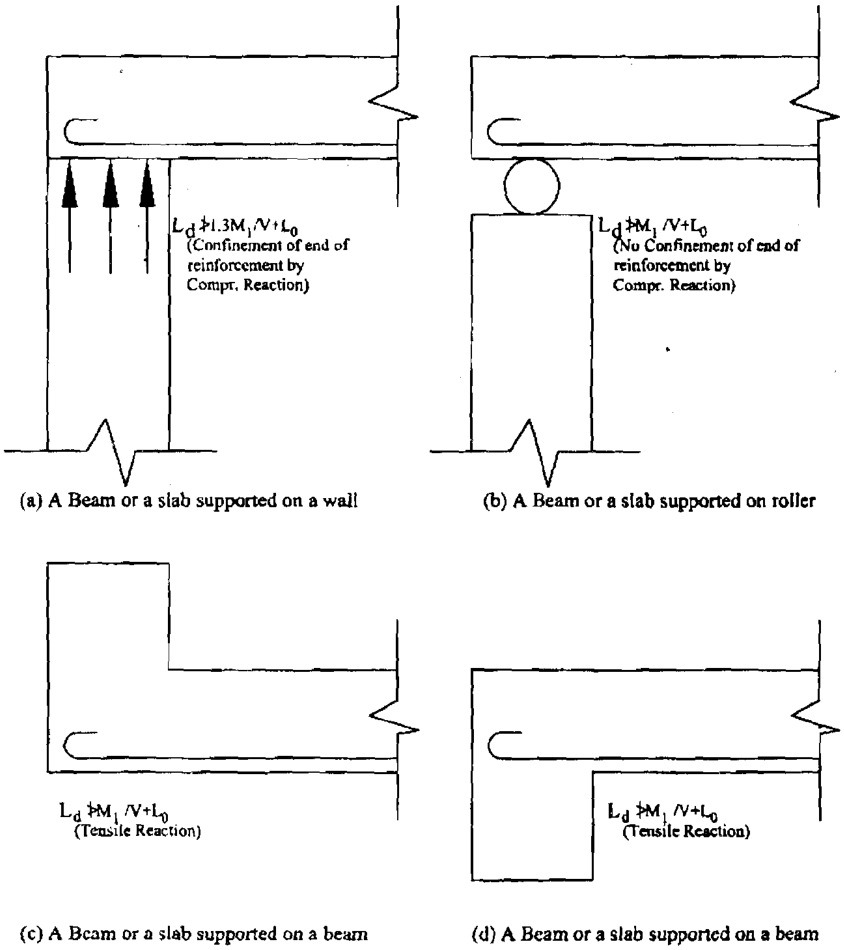


Figure 5.6 : Different Support Conditions for a Beam of a Slab

The bars may be extended beyond the centre of support in case of simple support by a length of L_o which may be counted in development length. L_o is the sum of anchorage length beyond the centre of the support and the equivalent anchorage value of any hook or mechanical anchorage at simple support. L_o cannot be of unlimited length and its values is limited by equation

$$L_d \ngtr \frac{M_1}{V} + L_0 \qquad \ldots (5.3)$$

To meet the requirement of inequality of the above equation 5.3, any one of the following three options may be adopted.

1) more bars may be provided for the supports so that M_1 is increased. Such arrangement may increase the reinforcement cost.

2) L_0 may be increased, but this cannot be done indefinitely, **owing to upper limit on L_0**, and

3) diameter of the main reinforcement may be reduced.

The third option is more suitable as it does not involve any extra cost.

$$\text{As} \qquad L_d \not< \frac{M_1}{V} + L_0$$

$$\text{or,} \qquad \frac{\phi f_d}{4\,\tau_{bd}} \not< \left(\frac{M_1}{V} + L_0\right)$$

$$\text{or,} \qquad \phi \not> \left(\frac{M_1}{V} + L_0\right)\frac{4\,\tau_{bd}}{f_d}$$

Above requirements are also applicable at section of *inflexion*, but the value of L_o in this case in limited to **12ϕ** or **d**, whichever is greater.

All about tensile development length at simple support for beams and slabs have been diagramatically represented in Figure 5.5.

The code further adds that the value of $\frac{M_1}{V}$ in Eq. (5.3) may be increased by 30% when ends of the reinforcement are confined by a *compressive reaction* (Figure 5.6).

SAQ 2

i) Define different types of bond.

ii) Derive expression for flexural bond and discuss as to why there is variation in actual bond stress and derived one?

iii) Define development length and find out its values.

iv) Explain codal provision of development length at simple support and at point of inflexion.

5.3 ANCHORING REINFORCING BARS

If sufficient length is not available on either sides of a critical section to develop design strength in a tensile reinforcement, end anchorages in the form of a bend or a hook may be provided to make it up.

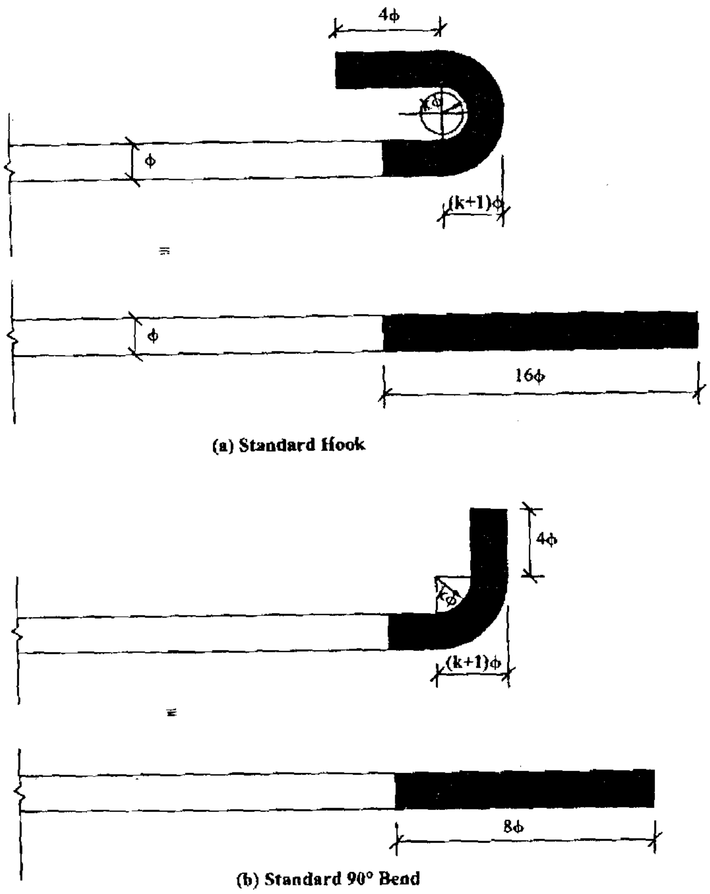
5.3.1 Anchoring Tension and Compression Reinforcement

The anchorage value of a bend shall be taken as four times the diameter of the bar for each 45° bend subject to a maximum of 16 times the diameter of the bar. The **standard bends** and **standard hooks** are shown in the Figure 5.7.

For bars in compression, the projected length of bends, hooks and straight bars beyond the bends shall be considered effective to be included in development length (Figure 5.8).

5.3.2 Anchoring Shear Reinforcements

The shear reinforcements are provided either in the form of a stirrup or bent up bars along with stirrups. The development length requirement for a stirrup deemed to have been satisfied if it is anchored round a bar of at least its own diameter for lengths beyond the curved portion of the anchorages shown in Figure 5.9.



k = 2 for mild steel &
k = 4 for cold worked steel

Figure 5.7 : Standard Hooks and Standard 90° Bend

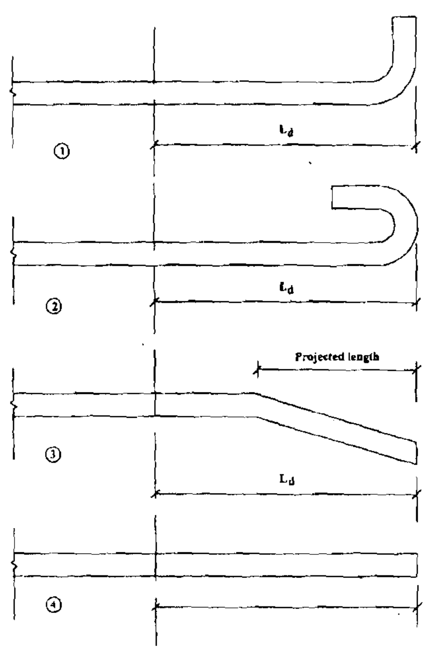
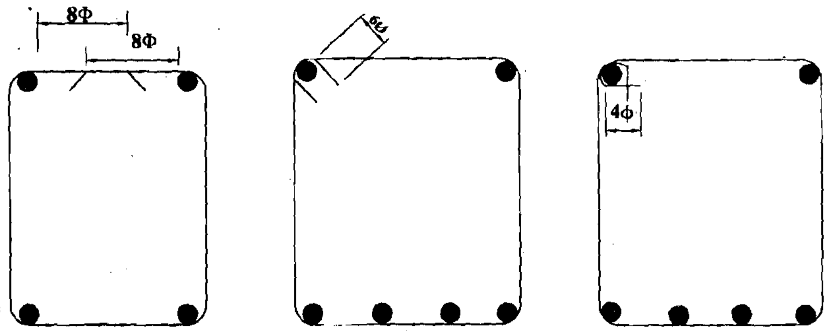


Figure 5.8 : Projected Lengths only to be Considered for L_d



8φ projection beyond curved portion of anchorage

6φ projection beyond curved portion of anchorage

4φ projection beyond curved portion of anchorage

Figure 5.9 : Anchorages for Shear Reinforcements

SAQ 3

Explain with sketches the anchoring of tension, compression and shear reinforcements.

5.4 CURTAILMENT OF TENSILE REINFORCEMENT IN FLEXURAL MEMBERS

For a flexural member, tensile reinforcements, at first, are provided at critical sections (i.e. at sections where maximum positive or negative bending moment occur). Generally bending moment varies along the length of a member, the maximum amount of tensile reinforcement need not be continued for the whole length and hence curtailment is necessitated. A bar which is no longer required to resist bending moment beyond a section may not be terminated abruptly at that section because of the fact that, if done so, this bar and/or continuing bars may not have adequate anchorage to develop full *design* strength. Therefore, a bar is continued beyond its theoretical cut-off point for a varying distances depending upon the location of section (i.e. whether the section is a simple support/end of a cantilever/point of inflexion / any other section). The continuance of a tensile reinforcing bar beyond theoretical cut-off point is also necessitated for variation of bending moment diagram due to positioning of live load along the span. At the point of termination of a reinforcing bar shear resisting capacity of that section diminishes and stress concentration occurs. Therefore, additional shear reinforcements are provided to cope with the above exigencies (problems).

Based upon the above considerations each type of section, where curtailment is done, has been explained with the help of examples in the following subsections.

5.4.1 Positive Moment Reinforcement at Simple Support

In Figure 5.10 let M_{max} be the maximum positive bending moment for which *six* reinforcing bars have been provided. As the area of reinforcements provided is generally more than that required, the moment capacity the section is *somewhat more* than M_{max}. If M_B is the moment of resistance of the beam section for four bars at a section ① - ① then two bars may be *theoretically* curtailed at this section. But the bars to be curtailed are extended beyond the theoretical cut off point by a length 12 ϕ or, d whichever is greater. To be more specific, if two bars may be theoretically curtailed at B, they are *actually* curtailed at C. Since the remaining four bars have to develop full *design* strength at the actual cut-off point, both sides of C must have at least L_d length.

Also towards r.h.s., the curtailed bars develop form zero at C to full *design* strength at D. Hence, the bending moment capacities of different section are shown by line ADC. Assuming further that another two bars may be theoretically terminated at E (section ②- ②); whereas they are actually curtailed at F. From point F only two bars (i.e. at least $\frac{1}{3}$ rd of the total positive reinforcements) are extended into the simple support along the same face of the beam equal to a length of at least $\frac{L_d}{3}$. If may be noted that HF, FC and CA are *greater* than or *equal to* L_d else *full* design strength will not develop in curtailed bars.

In addition, the requirement of $L_d \not> (\frac{M_1}{V} + L_0)$ at a section, say, near the support must be fulfilled as mentioned in subsection 5.2.3 and in Figure 5.10.

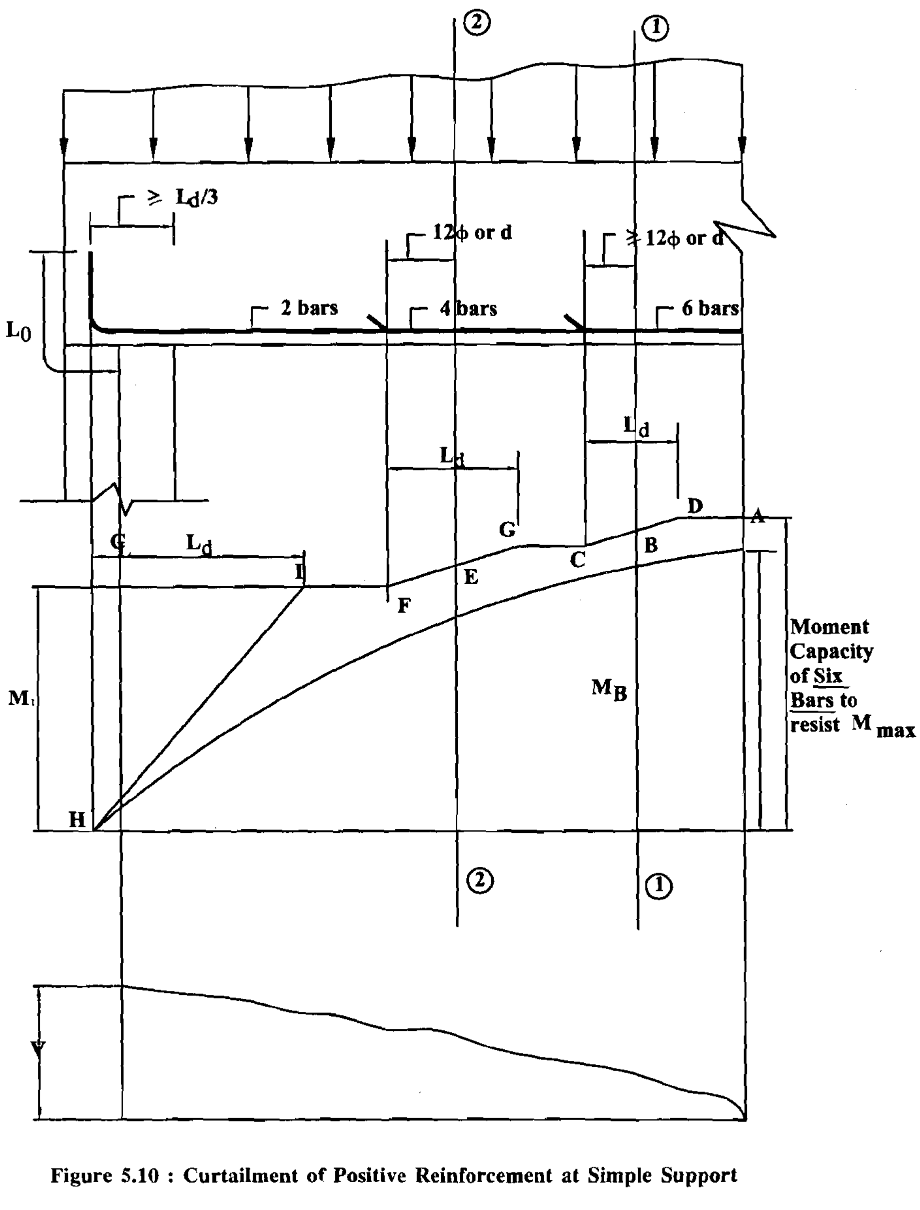


Figure 5.10 : Curtailment of Positive Reinforcement at Simple Support

5.4.2 Negative Moment Reinforcement of Fixed End of a Cantilever

Let *four* tensile reinforcing bars be provided to resist M_{max} (Figure 5.11). On both sides of A, the bars must extend to a length at least equal to L_d to develop full design strength. Two out of these four bars may be theoretically terminated at B; but they are extended to C (> 12 ϕ or d, whichever is greater).

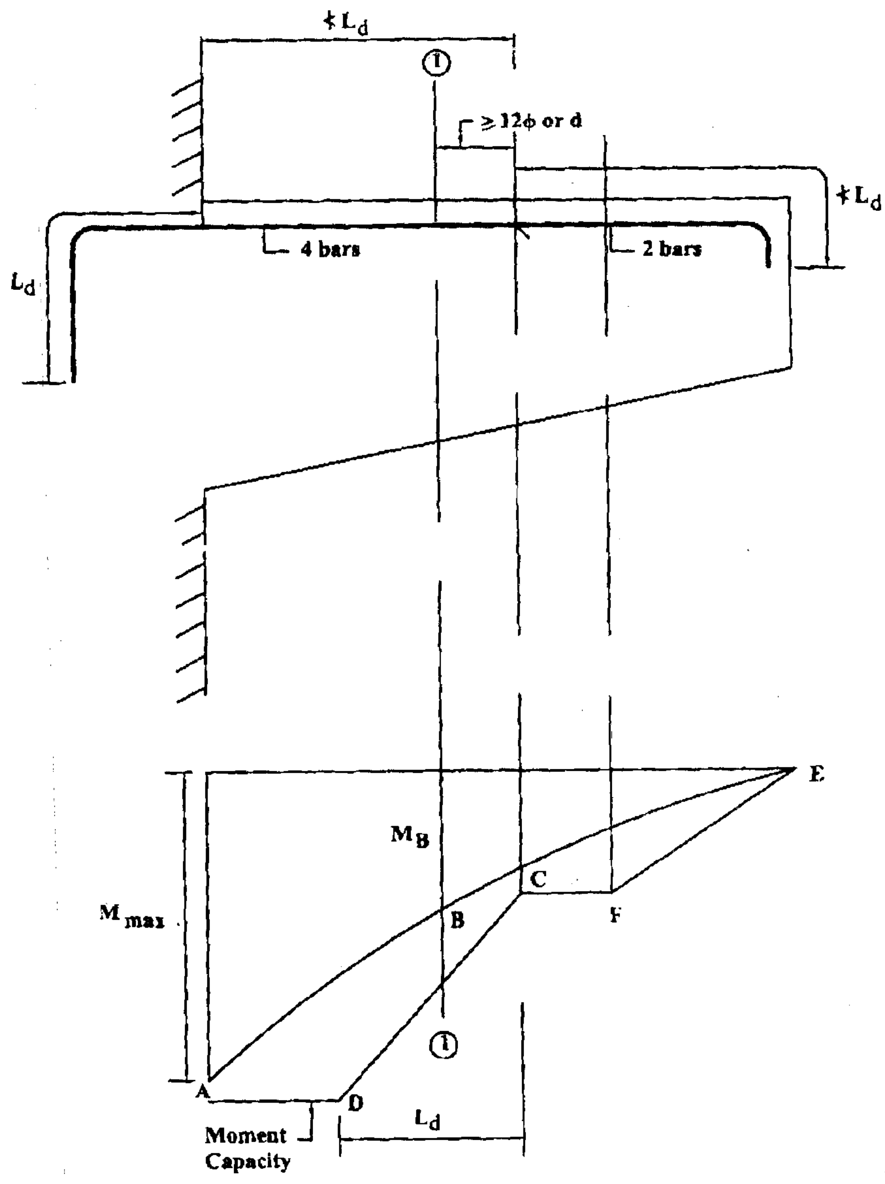


Figure 5.11 : Curtailment of Main Reinforcement of a Cantilever Beam

The continuing bars beyond C must be of length greater than L_d to develop full design strength.

5.4.3 Negative and Positive Moments Reinforcements at Continuous Edge

i) **Negative Reinforcement at Continuous Edge**

Let there be six bars provided for resisting *negative* bending moment at the face of the column support (Figure 5.12(a)) out of which four bars may be terminated theoretically at B; but they are extended to C (i.e. at least 12 ϕ or d, whichever is greater). *Two* bars (i.e. at least $\frac{1}{3}$ rd of the total *negative* reinforcement) are extended *beyond point of inflexion* by at least 12ϕ or d or $\frac{1}{16}$ th of the clear span whichever is greater. It may be noted that lengths AC and CE must be greater than L_d to develop *full design* strength at A and C respectively.

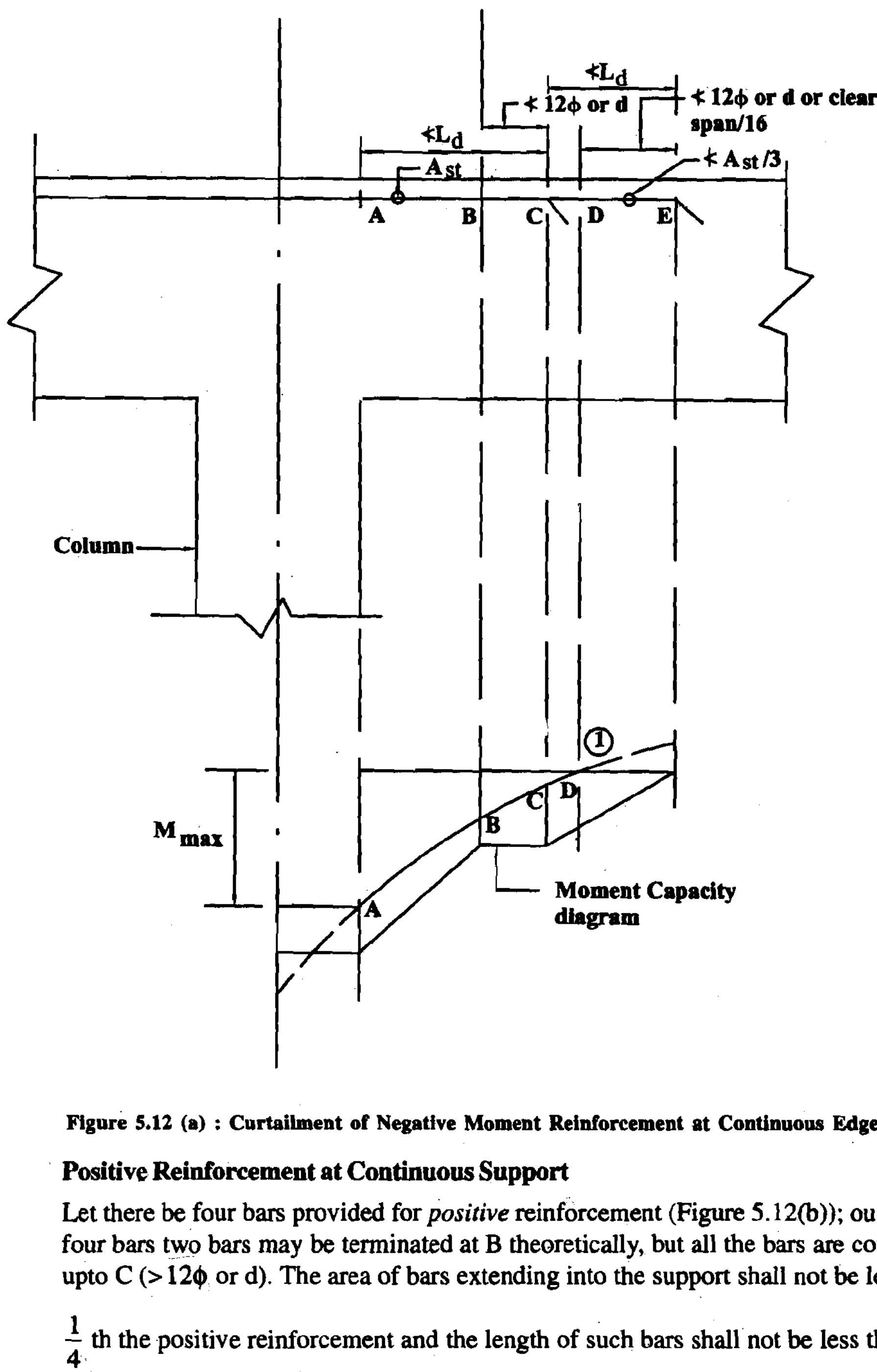


Figure 5.12 (a) : Curtailment of Negative Moment Reinforcement at Continuous Edge

ii) **Positive Reinforcement at Continuous Support**

Let there be four bars provided for *positive* reinforcement (Figure 5.12(b)); out of these four bars two bars may be terminated at B theoretically, but all the bars are continued upto C (> 12ϕ or d). The area of bars extending into the support shall not be less than $\frac{1}{4}$ th the positive reinforcement and the length of such bars shall not be less than $\frac{L_d}{3}$ beyond the inner face of the support. It may be noted that EC and CA shall be not less than L_d so that full *design* strength may develop at C and A respectively.

At point of inflexion, in addition to the above requirements, $L_d \leq \frac{M_1}{V} + L_0$ must be fulfilled.

Substituting $L_d = \dfrac{\phi f_d}{4 \tau_{bd}}$ in the above expression

$$\phi \leq \frac{4 \tau_{bd}}{f_d} \left[\frac{M_1}{V} + L_0 \right]$$

where M_1 = moment of resistance of the section assuming all reinforcement *at point of inflexion* to be stressed to f_d,

f_d = $0.87 f_y$ in case of limit state design and the permissible stress in the case of working stress design,

V = shear force *at the point of inflexion*, and

ϕ = dia of bars.

It may be noted that as the point of inflexion is not confined by compressive reaction, the value of $\dfrac{M_1}{V}$ is *not* increased by 30% and L_0 = actual embedment length of 12ϕ or d, whichever is greater, beyond the point of inflexion.

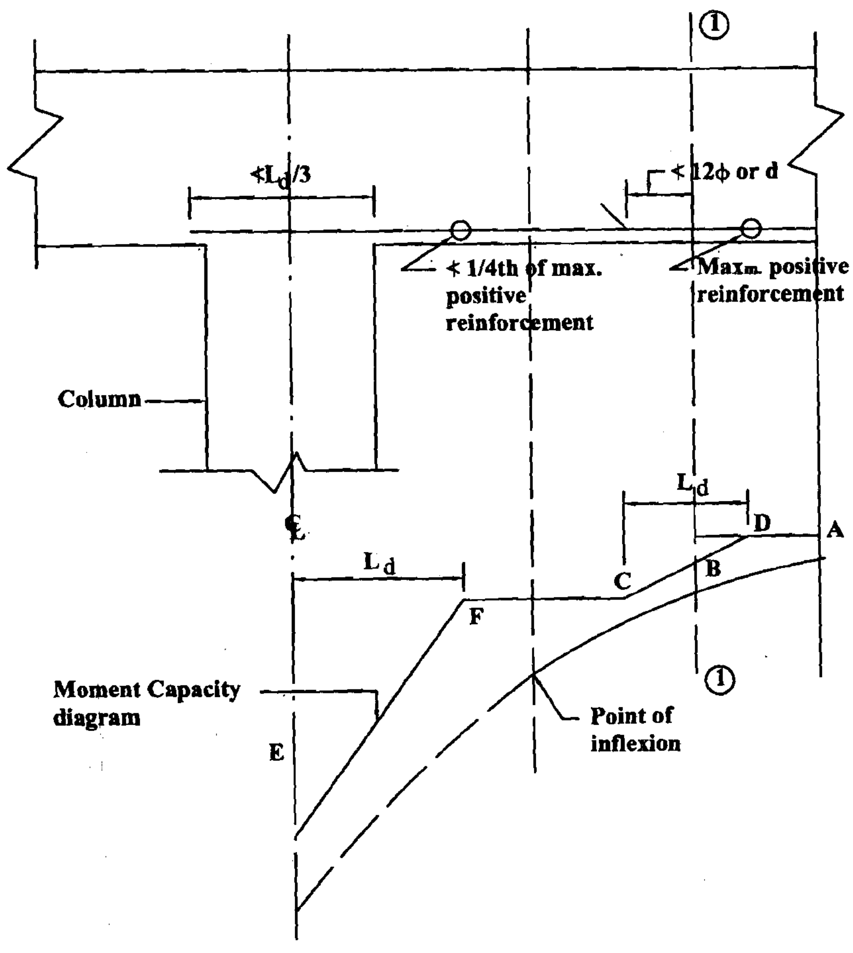


Figure 5.12 (b) : Curtailment of Positive Moment Reinforcement at Continuous Edge

5.4.4 Enhancement of Shear Strength of Cut-off Section

At a section where tensile reinforcements are curtailed, premature diagonal crack may develop if flexural stress in continuing bars and shear stress at that section are each near their maximum values. This tendency can be arrested by keeping either shearing stress low, or shear as well as flexural stresses low. Considering above facts, if *any one* of the following criteria is satisfied, the shear resisting capacity of the section can be safeguarded.

i) The shear at the cut-off section does not exceed $\frac{2}{3}$rd that permitted, including the shear capacity of web reinforcements. In terms of stresses, it can be expressed as $\tau_v \ngtr \frac{2}{3}(\tau_c + \tau_b)$

where τ_b = shear strength of web reinforcements.

ii) *Additional stirrup area* of not less than $\frac{0.4\,bs}{f_y}$ shall be provided along each terminated bar over a distance 0.75d from the cut-off section.

In other words, $A_{s,\,add} \nless \frac{0.4\,bs}{f_y}$

where $A_{s,\,add}$ = area of shear stirrups in excess of that required for shear and torsion at the cut-off section, and

$$s \ngtr \frac{d}{8\,\beta_\beta}\,;\ \left(\beta_b = \frac{\text{area of bars cut off}}{\text{Total area of bars at that section}}\right)$$

iii) Above mentioned provisions, additionally, strengthen cut-off section in shear. However, both bending as well as shear strengths of the cut-off section can be enhanced by providing continuing bars of double the area required for flexure and ensuring that the shear does not exceed three-fourth that permitted provided the tensile reinforcements bar diameter does not exceed 36 mm. Mathematically, for 36mm and smaller bars,

$$A_{st}' \nless 2\,A_{st}, \text{ and}$$

$$\tau_v \ngtr \frac{3}{4}(\tau_c + \tau_b)$$

where A'_{st} = Area of continuing bars at cut-off section, and

A_{st} = Area of tensile reinforcement required at cut-off section.

SAQ 4

i) Why a flexural reinforcing bar should be curtailed? Can they be terminated at actual cut-off points.

ii) Explain with sketch the curtailment of positive moment reinforcement in flexural member having simple supports?

iii) Explain with sketch the curtailment of negative reinforcement for a cantilever beam.

iv) Explain with sketches the curtailment of negative as well as positive reinforcements near continuous support.

5.5 SPLICING

Splicing of a reinforcing bar is necessitated where it cannot be provided as one continuous length. In other words, splicing is required to transfer force from one bar to another to maintain continuity of the bar spliced.

5.5.1 Lap Splices

Following rules may be observed in deciding the (i) suitable section for splicing (ii) methods of splicing and (iii) length of splicing.

i) **Suitable Section for Splicing**

Splices in flexure members should *not be* provided a) at sections where bending moment is more than 50% of the moment of resistance and ii) not more than half the bars shall be spliced at a section.

However, if due to unavoidable constraints, more than one-half of the bars are spliced at section or where splices are made at points of maximum stress, the strength of splicing is increased by increasing the lap length and/or using special or closely spaced stirrups around the length of the splice.

ii) **Lap Length of Splicing**

a) In flexural tension, lap length shall be taken equal to L_d (where $\sigma_s = f_d = 0.87f_y$) or 30ϕ whichever is greater.

b) For direct tension members lap length shall be $2L_d$ or 30ϕ whichever is greater and full spliced length shall be enclosed in spirals made of bars not less than 6 mm diameter bars having pitch not more than 100 mm. Hooks must be provided at the ends of the bars. However, the straight portion of the lap length shall be not less than 15 or 200 mm.

c) For compression members, lap length shall be equal to L_d or 24ϕ whichever is greater. Lapped splices for compression bars need not be staggered.

d) i) Centre to Centre distance between staggered lap slices shall not be less than 1.3 times L_d. L_d shall be calculated as described in (a) above.

ii) When bars of different diameters are spliced, the evaluation of lap length shall be made for bars of smaller diameter.

5.5.2 Splicing by Welding and Mechanical Connections

Welded splices or Machanical connections shall have design strength equal to 80% of the design strength of the bar for tension splices and 100% of the compression splices. However, 100% of the design strength may be assumed in tension when the spliced area forms not more than 20% of total area of steel at the section and the splices are staggered at least 600mm.

SAQ 5

Why splicing is needed? Explain different methods of splicing with sketches.

5.6 SUMMARY

Reinforced concrete being a composite material requires proper bond (shearing resistance) so that at a point the same compatible strains and deformations may develop at the interface of concrete and reinforcement.

Though bonds are of two types, flexural and anchorage, the latter one gives better estimate of actual bond stress. Sometimes when adequate length of a reinforcing bar is not available for development of full design strength, anchorage in the form of hooks and/or bends are provided. At point of curtailment (termination) of a reinforcing bar, *besides* providing development length (L_d) on either side of that section, shearing strength of that section is enhanced to take care of the fact that at such section the shearing strength of the beam is adversely affected.

Splicing of a reinforcing bar is needed where it cannot be provided in one continuous length. It may either be provided as *lap spicing*, where bars are lapped one over the other, or it may be accomplished either by welding or mechanical connections.

5.7 ANSWERS TO SAQs

SAQ 1

i) Refer Section 5.1

SAQ 2

i) Refer Sub-section 5.2.1

ii) Refer Sub-section 5.2.2

iii) Refer Sub-section 5.2.3

iv) Refer Sub-section 5.2.3

SAQ 3

Refer Section 5.3

SAQ 4

i) Refer Sub-section 5.4.1

ii) Refer Sub-section 5.4.2

iii) Refer Sub-section 5.4.3

iv) Refer Sub-section 5.4.4

SAQ 5

Refer Sub-section 5.5.3