UNIT 1 INTRODUCTION TO LIMIT STATE METHOD

Structure

1.1 INTRODUCTION

The discrepancies between the behaviour predicted by elastic analysis and that occurring in practice lead to employ for reinforced concrete design the theory that is one based on the conditions existing in an actual structure at various stages of loadings.

To be more specific, the Limit State Method of design of a R. C. Structure is based on limiting (i) the deflection and cracking at working state (loads) and (ii) stresses and strains at failure or collapse state (loads). Besides the above limiting states, a few structures performing special functions should comply with those limit states which are relevant to those structures.

Objectives

After going through this unit you will be able to apply the basic principles involved in the analysis and design of R. C. Structure through Limit State Method. Hence the objectives of this unit are as follows:

- Enumeration of various 'Limit States' considered while designing R. C. Structures,
- Determination of Design Values of stresses in concrete as well as in steel,
- Determination of Design Values of Loads at Collapse and at Serviceability Limit States,
- Determination of Permissible Deflection of Service State (Loads),
- Determination of Permissible width of Cracks in concrete at Service State (Loads), and
- Method of Analysis of Structures and their Components.

1.2 ENUMERATION OF VARIOUS LIMIT STATES CONSIDERED IN DESIGN

The fundamental concept of structural design is to produce structures that are safe and serviceable. The safety of a structure is ensured by limiting the values of stresses obtained by dividing the stresses in concrete and steel occurring at collapse of a structure at ultimate load by their respective factors called 'Partial Safety Factor' for materials. At the same time, at working (service) loads, the deflection and cracking, it any, of components of a structure are kept within prescribed limits.

Besides these two dominant 'Limit States' other limiting checks are there so that a structure may remain safe and does not become unfit while in use for which if was intended.

1.2.1 Limit State of Safety

The 'Collapse' of a structure or .any part of a structure means that the ultimate (collapse) loads are such as to produce excessive stresses, excessive deformations, rupture of critical sections, loss of equilibrium and stability demanding replacement of the structure.

Therefore, the 'Limit State of Safety' of a structure or of any of its parts means that the effects of collapse loads on a structure (i) in terms of stresses are limited to the values of its resisting capacity in bending, shear, torsion, axial loads and (ii) in terms of elastic or plastic instability, overturning etc. are limited to stability.

1.2.2 Limit State of Serviceability

A structure or any of its component must be fit for use at working (service) loads. At this load, excessive deflections and cracks may cause damage to the structure itself as well as to non-load bearing elements, affect the working of sensitive equipment, create fear psychosis, etc. causing discomfort to the users. Hence deflection in beams and cracking at critical sections of structural members are limited to such values that the serviceability is not adversely affected.

1.2.3 Other Limit States for Special Functions

Vibrations of structures at working loads, fatigue caused by cyclic loading of a machine foundation, lateral drift of tall structures under wind or earthquake loads, instability due to buckling of structural members, accidental loads due to vehicular traffic or minor blast and storage of materials injurious to reinforced cement concrete are but a few factors warranting other 'Limit States' to be imposed in addition to Limit States of Safety and Serviceability.

Hence special measures are adopted to safeguard against the damaging effects of any one or more of the above mentioned unusual factors on safety and serviceability of structures.

SAQ 1

i) Explain the term 'Limit States' and discuss various Limit States in Limit State Method of Structural Design of R. C. Structures.

1.3 DETERMINATION OF DESIGN VALUES FOR MATERIALS AND LOADS

1.3.1 General

Ideally the design stresses in materials and design loads should be the actual mean stresses obtained from test results and that the actual mean loads obtained from survey of loads respectively. But, even after controlled condition of manufacturing and testing, the values of stresses obtained from tests show variations. Similarly, inherent variations associated with the assessment of maximum loads is prohibitive in using such values.

Constructional faults, errors in mix proportion, inaccuracies in design assumptions and analysis, etc; the actual (characteristic) stresses in materials and that of loads are modified by another factor called "Partial Safety Factor" to achieve sensibly of same degree of safety as obtained by working stress method of design.

The characteristic stresses in materials are divided by 'Partial Safety Factors', whereas the characteristic values of loads are multiplied by Partial Safety Factors to achieve the above goal.

1.3.2 Design Values for Materials

To augment the shortcomings, mentioned above, the actual strength is taken as that value of strength of material below which, not more that 5% of the test results are expected to fall. The actual strength of a material so obtained are technically called "Characteristic Strength of Material".

The characteristic strength for concrete shall be as given in the Table 1.1 and, if necessary, modified for the age factor as per Table 1. 2.

Table 1.1 : Specified Characteristic Compressive Strength of Concrete at 28 Days

Grade Designation	Specified Characteristic Compressive Strength at 28 Days
(1)	(2)
	N/mm^2
M10	10
M15	15
M20	20
M25	25
M30	30
M35	35
M40	40

Note 1- In the designation of a concrete mix, letter M refers to the mix and the number to the specified characteristic compressive strength of 15-cm cube at 28 days, expressed in N/mm^2

Note 2- M 5 and M 7.5 grades of concrete may be used for lean concrete bases and simple foundation for masonry walls. These mixes need not be designed.

Note3- Grade of concrete lower than M 15 shall not be used in reinforced concrete.

Table 1.2 : Modification of Compressive Strength for Age Factors

Minimum Age of Member when Full Design Load/Stress is Expected (Months)	Age Factor
1	1.0
3	1.10
6	1.15
12	1.20

Note 1- No increase in respect of age at loading should be allowed where high alumina cement concrete is used.

Note 2- Where members are subjected to lower direct load during construction, they should be checked for stresses resulting from combination of direct load and bending during construction.

Note 3- The permissible stresses or design strength shall be based on the increased value of compressive strength.

Similarly Characteristic Strength for Steel is the minimum yield /0.2 percent proof stress specified in relevant Indian Standard Specifications. Generally, steel reinforcements having minimum yield/ 0.2% proof strengths of 250 MPa, 415 MPa and 500 MPa are in use.

These Characteristic Values for materials are further modified by 'Partial Safety Factors' to arrive at 'Design Values' used in designs.

Therefore, the Design Strength (values) of materials, f_d, is given by

$$f_d = \frac{f}{\gamma_m}$$

where,

f = characteristic strength of materials as per Table 1 & 2 for concrete and minimum yield /0.2% proof strength for steel.

γ_m = 'Partial Safety Factors' appropriate to the materials and the limit state considered.

For 'Limit State of Collapses' the values of Partial Safety Factors, γ_m, shall be 1.5 for concrete and 1.15 for steel.

1.3.3 Design Values for Loads

Due to unavailability of statistical data for each structure everytime, the characteristic loads are the loads as per Indian Standard Specifications depicted in Table 1.3.

Table 1.3 : Indian Standard Specifications for Loads

Characteristic Loads	Indian Standard Specification
Dead Loads	IS : 857 (Part 1) : 1987
Imposed Loads	IS : 857 (Part 2) : 1987
Wind Loads	IS : 857 (Part 3) : 1987
Seismic Loads	IS : 1893 : 1984

The 'Design Loads', f_d, for the 'Limit State of Collapse' as well as for the 'Limit State of Serviceability' are obtained by multiplying 'Characteristic Values' by appropriate 'Partial Safety Factors' as per Table 1.4.

Partial Safety Factors

The Values of γ_f given in Table 1.4 shall be used.

Table 1.4: Values of Partial Safety Factor γ_f for Loads

Load Combination	Limit State of Collapse			Limit States of Serviceability		
	DL	*LL*	*WL*	*DL*	*LL*	*WL*
(1)	(2)	(3)	(4)	(5)	(6)	(7)
DL+LL	1.5		-	1.0	1.0	-
DL+WL	1.5 or 0.9*	-	1.5	1.0	-	1.0
DL+LL+WL	1.2			1.0	0.8	0.8

NOTE 1- While considering earthquake effects, substitute *EL* for *WL*.

NOTE 2- For the limit states of serviceability, the values of γ_f given in this table are applicable for short term effects. While assessing the long term effects due to creep, the dead load and that part of the live likely to be permanent may only be considered.

* This value is to be considered when stability against overturning or stress reversal is critical.

In other words,

$$F_d = F\,\gamma_f$$

where,

F = Characteristic Load (Table 1.3), and

γ_f = 'Partial Safety Factors' appropriate to the type of loadings and the limit state being considered (Table 1.4)

SAQ 2

i) Write short notes on:

a) Characteristic Strength

b) Characteristic Loads, and

c) Partial Safety Factors.

ii) For maximum sagging bending moment at support in a continuous R. C. Beam, live load should be placed on

a) spans adjacent to the support plus alternate spans

b) all the spans except the spans adjacent to the supports

c) spans next to the adjacent spans of the support plus alternate spans

d) spans adjacent to supports only.

1.4 DETERMINATION OF DESIGN VALUES FOR DEFLECTION AND CRACKING

1.4.1 General

The deflection in beams and slabs and cracking at critical sections of structural elements must be within the Design Values prescribed for the same.

1.4.2 Design Values for Deflection

The final deflection due to all loads including the effects of temperature, creep and shrinkage from as cast level shall not exceed $\frac{\text{span}}{250}$.

A part of the above mentioned deflection caused by partitions or application of finishes including the effects of temperature, creep and shrinkage shall be not more than $\frac{\text{span}}{350}$ or 20 mm whichever is less.

However, in normal circumstances, the calculation of deflection for flexual members is laborious and many of the parameters required for such calculations are not known precisely at the design stage, alternative simpler approach for control of deflection envisaged is as follows:

The control of deflection aspect deemed to have been satisfied if the following criteria with modification factors given as under are fulfilled.

i) For rectangular beams and for slabs* upto 10 m span the 'Basic Values' for $\frac{\text{span}}{\text{effective.depth}}$ ratio are not greater than as given Table 1.5.

Table 1.5 : Basic Values of $\frac{l_{ef}}{d}$ for Different Support Conditions

Support Conditions	$\frac{\text{span}}{\text{effective depth}}\left(\frac{l_{ef}}{d}\right)$
Cantilever	7
Simply Supported	20
Continuous	26

The Basic Values of $\frac{l_{ef}}{d}$ are further modified for different parameters as depicted in Table 1.6.

* For two-way slabs,

i) the shorter of the two spans should be used for calculating the spans to effective depth ratios; and

ii) a) if shorter span is upto 3.5,

b) the slab is reinforced with mild steel, and

c) with loading class upto 3 kN/m^2; the span to overall depth ratio given below may be assumed to have satisfied the vertical deflection limits:

Supply supported slabs	35
continuous slab	40

iii) if the slabs mentioned in (ii) are reinforced with deformed bars of Grade Fe415 the values given above should be multiplied by 0.8

Table 1.6 : Modification Factors for $\frac{l_{ef}}{d}$ Ratio for Different Parameters

Parameters		Modification (Multiplication) Factors
(a)	$l_{ef} > 10m$ *	$\frac{10}{l_{ef}}$
(b)	Area and type of tensile reinforcement	Figure 1.1
(c)	Area of compression reinforcement	Figure 1.2
(d)	Ratio of $\frac{b_w}{b_f}$ for flanged beams. **	Figure 1.3

* For cantilever beams of greater span than 10 m deflection calculation shall be made as per Appendix B of the code.

** Reinforcement % shall be based on area of section equal to b.d

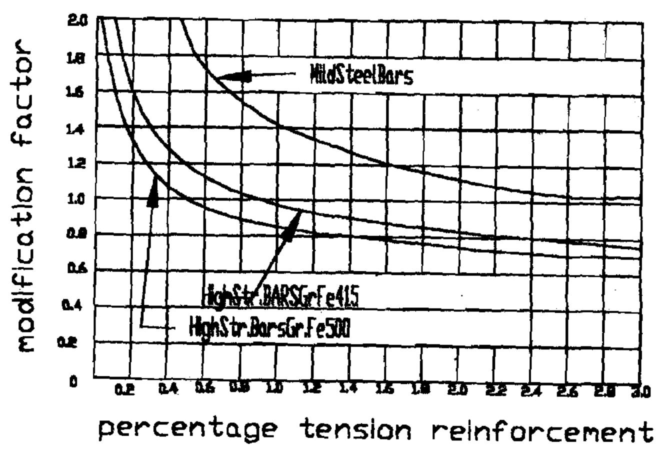

Figure 1.1 : Modification Factor for Tension Reinforcement

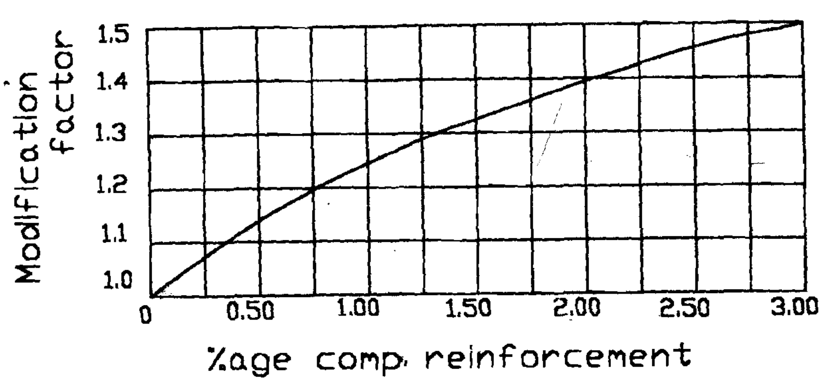

Figure 1.2 : Modification Factor for Compression Reinforcement

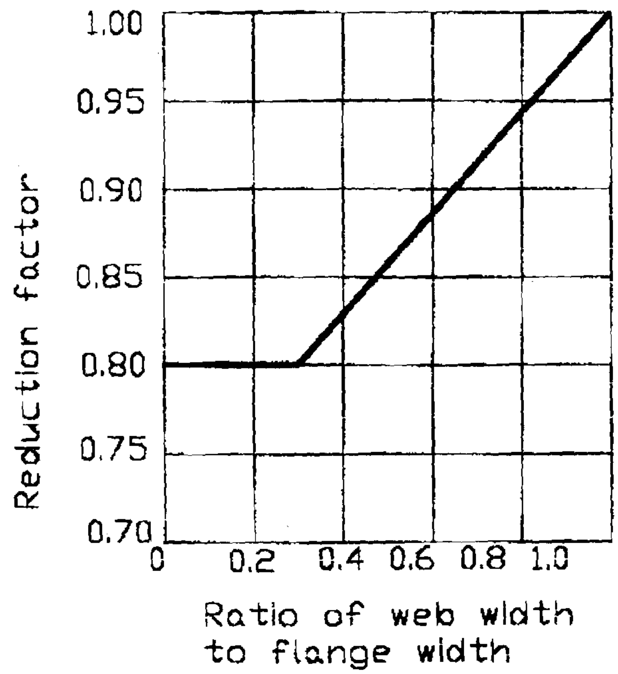

Figure 1.3 : Reduction Factors for Ratios of Span to Effective Depth for Flanged Beams

1.4.3 Design Values for Cracking

The evaluation of actual crack width at service load has not been perfected, hence it is measured with the help of strips of different thicknesses made for this purpose. For flexural members the actual (design) crack width for a structural member is limited to 0.3 mm from aesthetic consideration as well as for protection against mild exposure condition for tensile reinforcements. For 'severe' exposure conditions to tensile reinforcements, the surface cracks are limited to 0.004 times the nominal cover. For columns, if the axial load is more than

$0.2\, f_{ck}A_c$, cracks due to bending need not be checked, but where the axial load is less than

$0.2\, f_{ck}A_c$, the crack width limitations are the same as those for flexural members.

Depending upon the special functional requirements of a structure the acceptable crack limit of cracks are fixed as per relevant Indian Standard Specifications. For example:

i) For bins and silos the crack width may go upto 0.2 mm but where water tightness is required it is limited to 0.1 mm.

ii) For liquid storage tanks the crack widths are limited by limiting the permissible stresses of concrete and steel to lower value prescribed in relevant IS specifications.

To restrict the crack widths within permissible limits under normal exposure conditions flexural members are provided with tensile reinforcement having maximum spacing as follows:

i) For Beams

The horizonal distance between parallel bars on tension face of a beam shall be provided as per Table 1.7 depending upon the characteristic strength of concrete and amount of redistribution of members carried out in analysis.

Table 1.7 : Maximum Clear Distance between Bars

f_y	Percentage Redistribution To or From Section Considered				
	– 30	–15	0	+15	+30
	Maximum Clear Distance between Bars				
N/mm²	mm	mm	mm	mm	mm
250	215	260	300	300	300
415	125	155	180	210	235
500	105	130	150	175	195

Note- For aggressive environments the spacing given in the above table may be applicable only when f_y has been limited to 300 N/mm² in limit state design and σ_{st} limited to 165 N/mm² in working stress design.

Where the depth of beams exceed 750 mm, side reinforcements are provided on two faces to control crack width. These reinforcements are 1% of the web area equally distributed to both faces on a spacing not exceeding 300 mm or web thickness whichever is less.

ii) For Slabs

The maximum horizontal distance between main reinforcing bars shall not be more than 3 times the effective depth or 450 whichever is less.

The maximum horizontal distance between distribution bars shall not be more than five times the effective depth or 450 whichever is smaller.

In flat slab construction, the maximum spacing of bars shall not be more than two times the thickness of slab.

For ribbed, hollow block or voided simply supported slabs, the same rules as those for solid slabs are applicable. The continuous ribbed, hollow block or voided slabs are designed as simply supported ones because of difficulties in accommodating reinforcements at supports. Therefore, to control the cracking at continuous supports the reinforcement at the supports should not be less than one quarter that required in the middle of adjoining spans and shall extent at least one tenth of the clear span into adjoining spans.

SAQ 3

i) Limit State of Serviceability for deflection including the effect for creep, shrinkage and temperature occurring after erection of partitions and application of finishes as applicable to floors and roofs is restricted to

a) $\frac{\text{Span}}{150}$ b) $\frac{\text{Span}}{200}$

c) $\frac{\text{Span}}{250}$ d) $\frac{\text{Span}}{350}$

ii) In case of 2-way slab, the limiting deflection of the slab is

a) primarily a function of the long span,

b) primarily a function of short span,

c) independent of long or shorts span, and

d) dependent on both long and short spans.

iii) Deflection can be controlled by using the appropriate

a) aspect ratio b) modular ratio

c) $\frac{\text{Span}}{\text{depth}}$ ratio d) water cement ratio

iv) In the limit state approach spacing of main reinforcement controls primarily

a) collapse b) cracking

c) deflection d) durability

v) Side face reinforcement is provided in a beam when the depth of web exceeds

a) 300 mm b) 450 mm

c) 500 mm d) 750 mm

vi) A reinforced cantilever beam, span 4m, has a cross section of 150 X 500 mm. If checked for lateral stability and deflection, the beam will

a) fail in deflection only,

b) fail in lateral stability,

c) fail in both deflection and lateral stability, and

d) satisfy the requirements of deflection and lateral stability.

1.5 ANALYSIS OF STRUCTURE FOR DESIGN

For both methods of design-Working Stress Method and Limit State Method-the code prescribes analysis of structure for their respective design loads by Linear Elastic Theory. However, a redistribution of moments for flexural members are done to bring the values of moments and shear forces closer to actual ones produced by design loads.

Basic inputs for analysis of a structure are geometrical & material properties as well as loading arrangement for live loads for design. In this section only the most basic properties, which are common to all types of structures will be described. The other relevant properties will be dealt with respective types of structures or structural members to be designed.

1.5.1 Unit Weight of Concrete

The unit weights of plain concrete and reinforced concrete made of sand and gravel or crushed natural aggregates may be taken as 24 kN/m^3 and 25kN/m^3 respectively.

1.5.2 Stiffness

Stiffness is one of the geometrical properties of a structural element required for analysis of a structure. In general, it is the amount of force/moment required to cause unit deformation. The stiffness of tension or compression member is $\alpha \frac{EA}{\ell}$ and that for flexural member is $\alpha' \frac{EI}{\ell}$

where,

α & α' are coefficients depending on end conditions of the members

E = Modulus of Elasticity of concrete,

A = Equivalent area of cross section,

I = Moment of inertia of cross section, and

ℓ = Effective length of member.

The torsional rigidity stiffness of a member is given by $\frac{CG}{\ell}$

where,

C = value of polar moment of inertia modified by a constant for torsional member, and

G = Modulus of rigidity of concrete.

1.5.3 Modulus of Elasticity

Modulus of Elasticity of concrete is related to characteristic strength of concrete and is given as

$$E_C^* = 5700 \sqrt{f_{ck}}$$

where,

E_c = Short term static modulus of elasticity in (N/mm²), and

f_{ck} = Characteristic strength of concrete (N/mm²).

1.5.4 Moment of Inertia

In elastic analysis of indeterminate structures due to design loads relative stiffness of members are used. The code recommends that the moment of inertia of a section may be calculated by either of the three methods :

i) by considering total cross section of the member ignoring steel,

ii) by taking cross section of concrete plus the area of reinforcement transformed on the basis of modular ratio, or

iii) by taking area of concrete in compression plus the transformed area of steel based on modular ratio.

Only one of the three methods for calculating moment of inertia must be used for all members of a particular structure in elastic analysis.

1.5.5 Redistribution of Moments in Continuous Beams & Frames

Redistribution normally means reduction. Now the question arises as to why this reduction of moment is desired at certain sections of a beam. As per IS Code all structures are analysed by the linear elastic theory to calculate the internal forces and moments for the applied design loads at limit state of collapse. Accordingly, it is found that support hogging moments are much more than span moment resulting in congestion of reinforcement at supports and necessitating bigger cross section or doubly reinforced sections at supports leading to uneconomical design.

As a matter of fact, at Limit State of Collapse, neither linearly elastic analysis nor purely plastic analysis, as for steel structures, are justified for reinforced concrete structures. A reinforced concrete member under loading shows non-linear behaviour from the very beginning

For simplicity of analysis the moment - curvature relation is idealised as bilinear. But again due to limiting maximum strain of 0.0035 in concrete the critical section cannot have infinite rotation/curvature at constant maximun moment as in the case of steel justifying purely plastic analysis. Thus frames or indeterminate beams at collapse stage are analysed elastically and the

* E_c is used only for calculation of short term deflection (i.e. deflection due to service loads) and not for any other purpose.

moments at critical sections are modified to represent the conditions at failure stage under design ultimate loads. The modified moments at collapse for different cross section must also satisfy the equilibrium conditions and deformation criteria. Thus Indian Standard has codified the limits of redistribution of moments and of rotation of plastic hinges at ultimate moments keeping equilibrium of structure in view.

Explanatory Example*

The redistribution is better understood through the example given below.

Let us consider a propped cantilever beam loaded with design ultimate load, w_u ,at Limit State of collapse Figure 1.4 (a).

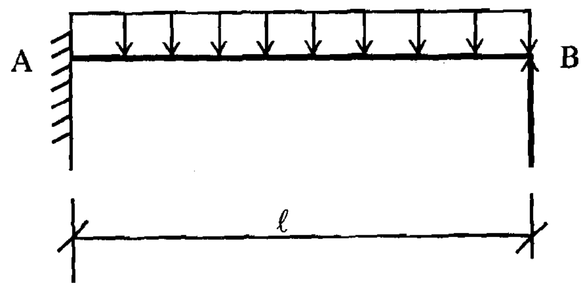

Figure 1.4 (a) : Beam with Loading

The B.M.D. obtained by linear elastic analysis is shown in Figure 1.4 (b).

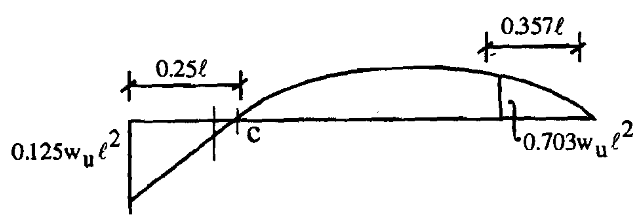

Figure 1.4 (b) : B.M.D. by Elastic Analysis

If a maximum redistribution of 30% is made for support moment the resulting B.M.D. is as shown in Figure 1.4 (c).

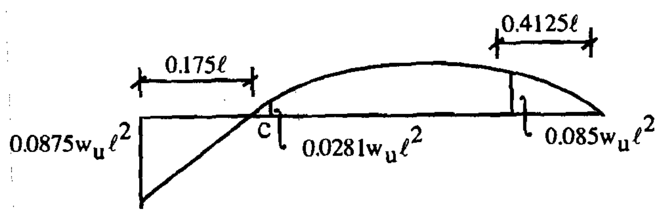

Figure 1.4 (c) : B.M.D. after 30% Redistribution of Moment at Fixed Support

Again the points of contraflexure before and after redistribution are 0.25ℓ and 0.175ℓ from L. H. support as shown in Figure 1.4 (b) & (c) respectively.

* Explanatory Calculations on Page 17

The bending moment at C after redistribution is 0.0281 $w_u \ell^2$. The Code says that the section at C must have a moment of resistance equal to 70% of 0.0281 $w_u \ell^2$ i.e. 0.0197 $w_u \ell^2$.

To ensure the rotational capability of any section where the moment capacity after redistribution is less than that from elastic maximum moment diagram the following relation must be satisfied.

$$\frac{x_u}{d} + \frac{\delta M}{100} \leq 0.6$$

where,

x_u = depth of neutral axis,

d = effective depth, and

δM = % variation in moment.

In the above equation δM may have maximum value of 30% i.e. maximum $\frac{\delta M}{100}$ may be 0.3; whereas maximum value of $\frac{x_{u\,max}}{d}$ for all grades of steel is around 0.5 as may be seen from the code. Now if $\delta M \left(\text{i.e.} \frac{\delta M}{100} = 0.3 \right)$ is maximised $\frac{x_u}{d}$ may have the maximum value of 0.3 . If $x_u \ll x_{u\,max}$, the maximum curvature $\left(\frac{0.0035}{x_u} \right)$ at collapse will be larger ensuring greater rotational capacity.

*** Explanatory Calculations**

I. From compatibility condition at B, vertical deflection at B = 0 (Figure I)

In other words,

$$\Delta_{B1} + \Delta_{B2} = 0$$

$$\text{or, } \frac{w_u \ell^4}{8EI} + \frac{R_B \ell^3}{3EI} = 0$$

$$\text{or, } R_B = \frac{3}{8} w_u \ell$$

Again from equilibrium condition

$$\Sigma M_A = 0$$

$$\text{or, } -M_A + \frac{w_u \ell^2}{2} - \frac{3}{8} w_u \ell^2 = 0$$

$$\text{or, } M_A = 0.125 w_u \ell^2$$

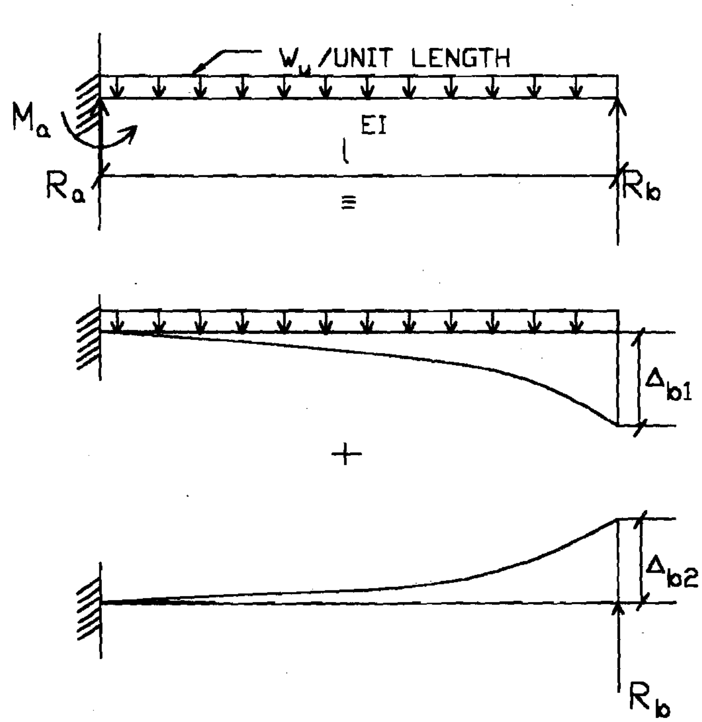

Figure I

Let maximum span bending moment occurs at x from R.H.S. Therefore, SF at x will be zero i.e.

$$\frac{3}{8}w_u\ell - w_u x = 0; \text{ or, } x = \frac{3}{8}\ell$$

$$\therefore \; +M_{max} = \frac{3}{8}w_u\ell \times \frac{3}{8}\ell - \frac{w_u}{2}\left(\frac{3}{8}\ell\right)^2 = 0.0703w_u\ell^2$$

Let point of contraflexure lie at x from L.H.S.

Therefore,

$$0.625w_u\ell x - 0.125w_u\ell^2 - w_u\frac{x^2}{2} = 0$$

$$\text{or, } 0.5x^2 - 0.625\ell x + 0.125\ell^2 = 0$$

$$\text{or, } x = \frac{0.625\ell \pm \sqrt{(0.625\ell)^2 - 4\times 0.5\times 0.125\ell^2}}{2\times 0.5} \text{ or, } x = 0.625\ell \pm 0.375\ell = 0.25\ell$$

II. Calculation for B.M.D (Figure II) after 30% redistribution of moment at fixed end.

$$M'_A = 0.125w_u\ell^2 \times \frac{70}{100} = 0.0875w_u\ell^2$$

From equilibrium conditions

$$\Sigma M'_A = 0$$

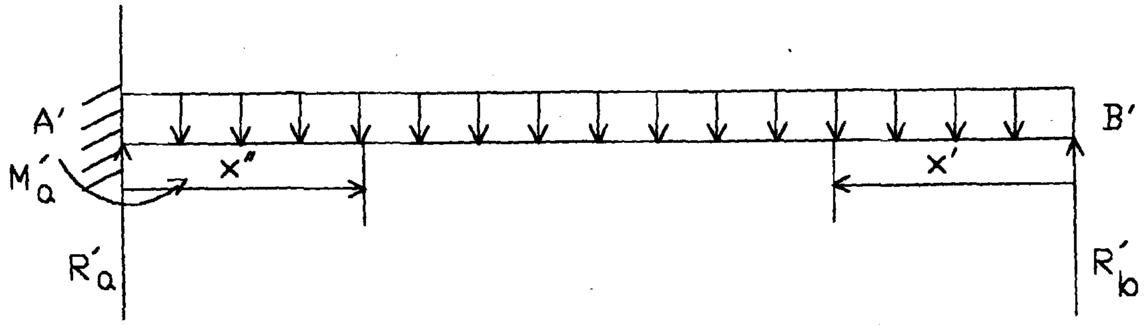

Figure II

or, $-0.0875 w_u \ell^2 + w_u \ell^2 - R'_B \ell = 0$

$R'_B = 0.4125 w_u \ell$

$R'_A = w_u \ell - 0.4125 w_u \ell = 0.5875 w_u \ell$

Let maximum span bending moment occurs at x' from R.H.S. Therefore, S.F. will be zero at x', i.e.

$0.4125 w_u \ell - w_u x' = 0$ or, $x' = 0.4125\ell$

$\therefore +M_{max} = 0.4125 w_u \ell \times 0.4125\ell - \frac{w_u}{2} \times (0.4125\ell)^2 = 0.085 w_u \ell^2$

Let point of contraflexure be at x" from L.H.S. Therefore,

$$-0.0875 w_u \ell^2 + 0.5875 w_u \ell x'' - \frac{w_u x''^2}{2} = 0$$

or, $0.5 x''^2 - 0.5875 \ell x'' + 0.0875 \ell^2 = 0$

or, $x'' = \frac{0.5875\ell \pm \sqrt{(0.5875\ell)^2 - 4 \times 0.5 \times 0.0875}}{2 \times 0.5}$ or, $x'' = 0.5875\ell \pm 0.4125\ell = 0.175\ell$

III. B.M. at C in Figure II i.e. at 0.75ℓ from B'

$$M_c = 0.4125 w_u \ell \times 0.75\ell - \frac{w_u}{2}(0.75\ell)^2$$

or, $M_c = 0.0281 w_u \ell^2$

SAQ 4

i) Explain the term Redistribution of Moments in continuous Beams and Frames with example.

1.6 SUMMARY

i) A R. C. structure when designed according to Limit State Method is safe and serviceable during its lifetime. Safety means that when the structure will be under design loads (collapse loads) the stresses in steel and concrete will not exceed their respective design strengths. Design load equals Characteristic

Load multiplied by Partial Safety Factor appropriate to the nature of loading and the limit state being considered; whereas Design strength equals Characteristic Strength divided by Partial Safety Factor appropriate to material and limit state being considered.

Similarly, Limit State of Serviceability demands that under Design loads (working loads) the deflections and cracks in any member shall be within permissible limits prescribed for the same.

ii) A R. C. structure will be analysed for Design loads (collapse loads) by Linear Elastic Theory. The B. M. D. so obtained should be modified by redistributing them as prescribed in the code.

1.7 ANSWERS TO SAQs

SAQ 1

i) Refer Section 1.1 & 1.2

SAQ 2

i) Refer Section 1.3

SAQ 3

i) d)

ii) b)

iii) c)

iv) b)

v) d)

vi) e) Deflection check. As per clause 22.2.1 of IS: 456-1978

$$\frac{\ell}{D} = \frac{4}{0.5} = 8 > 7$$

The beam will fail in deflection

Lateral Stability check. As per clause 22.3 IS: 456- 1978

$25 \times 0.15 = 3.75 < 4\text{m}.$

and $\dfrac{100 \times 0.15^2}{(0.5 - 0.04)} = 4.8 > 4\,\text{m}$

The beam also fails in lateral stability.

Therefore, the beam will fail in both deflection and lateral stability.

SAQ 4

(i) Refer Subsection 1.5.6