Design of Concrete Structure

UNIT 1
Concrete properties and WSM

Concrete making materials

Concrete is a composite material composed of Coarse aggregate, fine aggregate (sand) cement and water.

a building material made from a mixture of broken stone or gravel, sand, cement, and water, which can be spread or poured into moulds and forms a stone-like mass on hardening

Constituents of concrete

Cement

A **cement** is a binder, a substance used in construction that sets, hardens and adheres to other materials, binding them together.

Coarse Aggregate

Coarse aggregates are particles greater than 4.75mm, but generally range between 9.5mm to 37.5mm in diameter. They can either be from Primary, Secondary or Recycled sources

Fine Aggregate

Fine aggregates are the aggregates whose size is less than 4.75 mm. Example: Sand is used as fine aggregate in the preparation of concrete and cement mortar

Water

Almost any natural water that is drinkable and has no pronounced taste or odor may be used as mixing water for concrete. Excessive impurities in mixing water not only may affect setting time and concrete strength, but can also cause efflorescence, staining, corrosion of reinforcement, volume instability, and reduced durability

Properties of concrete and reinforcements

Concrete properties

Concrete has relatively high compressive strength, but significantly lower tensile strength, and as such is usually reinforced with materials that are strong in tension (often steel). The elasticity of concrete is relatively constant at low stress levels but starts decreasing at higher stress levels as matrix cracking develops.

Properties of concrete and reinforcements Contd..

Reinforcement properties

Several types of reinforcement have been used in CRCP, but by far the most common is deformed steel bars. Other innovative materials include solid stainless steel, stainless steel clad, and other proprietary materials such as fiber reinforced polymer (FRP) bars.

Testing of concrete

Following are the various test performed on concrete

Test on workability

- Slump test
- Compacting factor test
- Flow table test
- Vee bee consistometer method

Test on compressive strength

Test on flexural strenght

Test on modulus of elasticity

Test on fresh concrete

Test cement content of hardened concrete

Introduction to Various Design Philosophies

Their are three philosophies for the design of reinforced concrete

Working Stress Method

Ultimate Strength Method

Limit State method

UNIT 2
INTRODUCTION OF LSM

Assumptions in Limit State Design Method

1. Plane section normal to the axis remains plane even after bending. i.e. strain at any point on the cross section is directly proportional to the distance from the N.A.
2. Maximum strain in concrete at the outer most compression fibre is taken as 0.0035 in bending.
3. The tensile strength of the concrete is ignored.
4. The relationship between the compressive stress distribution in concrete and the strain in concrete may be assumed to be rectangle, trapezoidal, parabola or any other shape.
5. The stresses in the reinforcement are derived from representative stress – strain curve for the type of steel used.
6. The maximum strain in tension reinforcement in the section at failure shall not be less than

$$f_y / (E_s * 1.15) + 0.002$$

Balanced/ Under-reinforced/ Over-Reinforced Sections

Balanced Sections

Section in which, tension steel also reaches yield strain simultaneously as the concrete reaches the failure strain in bending

Under Reinforced Section

Section in which, tension steel also reaches yield strain at loads lower then the load at which concrete reaches the failure strain in bending

Over Reinforced Section

Section in which, tension steel also reaches yield strain at loads higher then the load at which concrete reaches the failure strain in bending

Design of Rectangular Singly Reinforced beams Limit State Design Method

Problem Type 1

Identify the type of section, balance, under reinforced or over reinforced

Data Given

Identify the type of section, balance, under reinforced or over reinforced

Procedure

If Xu/d = Xu,max/d Balanced

If Xu/d < Xu,max/d Under Reinforced

If Xu/d > Xu,max/d Over Reinforced

Fe	Xu,max/d
250	0.53
415	0.48
500	0.46

Xu/d = (0.87*fy*Ast)/ (0.36*fck*b*d

As per IS:456-2000, p-69

Problem Type 2

Calculate of Moment of resistance

Data Given

Grade of Concrete & Steel, Size of beam & Reinforcement provided

Procedure

1) If Xu/d = Xu, max /d **Balanced**

M.R. = Mu,lim = 0.36 * Xu, max /d* (1 - 0.42 Xu, max /d) b * d^2 * fck

2) If Xu/d < Xu, max /d **Under Reinforced**

M.R. = Mu = 0.87 * fy * Ast * d * (1 - fck*d*b fy*Ast) OR

M.R. = Mu = 0.87 * fy * Ast * d * (1 – 0.42 Xu/d)

3) If Xu/d > Xu, max /d **Over Reinforced** **Revise the depth.**

Problem Type 3

Design the beam. Find out the Depth of Beam 'D' & Reinforcement Required 'Ast'

Data Given

Grade of Concrete & Steel, Width of beam, B.M. or loading on the beam with the span of the beam

Procedure

We have to design as a 'Balanced Design' For finding 'd', use the equation,

M.R. = Mu,lim = 0.36 * Xu,max/d * (1 - 0.42 Xu,max/d) b * d^2 * fck

For finding 'Ast', use the equation, M.R. = Mu = 0.87 * fy * Ast * d * (1 - fck*d*b fy*Ast) OR M.R. = Mu = 0.87 * fy * Ast * d * (1 – 0.42 Xu/d)

Where,

d = effective depth of beam in mm

b = width of beam in mm

Xu = depth of actual neutral axis in mm from extreme compression fibre

Xu,max = depth of critical neutral axis in mm from extreme compression fibre

Ast = area of tensile reinforcement

fck = characteristic compressive strength of concrete in Mpa

fy = characteristic strength of steel in MPa

Mu,lim = limiting M.R. of a section without compression reinforcement

UNIT 3
Shear Reinforcement

Behaviour of RC beam in Shear

Modes of failure

Diagonal Tension Failure

Flexural shear failure

Digonal Compression Failure

Shear Strength of beams with and without shear reinforcement

Without Shear Reinforcement

The design shear strenght τ_c in beam without shear reinforcement based on the percentage of longitudinal reinforcement and grade of concrete.

Value of τ_c for 0.20 % steel for different grades of concrete is shown in table

100*A_{st}	Grade of concrete					
	M15	M20	M25	M30	M35	M40
0.20	0.32	0.32	0.33	0.33	0.35	0.34

Minimum and Maximum shear reinforcement

Maximum 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 45^0, where *d is the effective depth of the section* under consideration. In no case shall the spacing exceed 300 mm.

Minimum shear reinforcement

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

$$A_{sv} / bS_v = 0.4/0.87f_y$$

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 web of flanged beam

f_y = characteristic strength of the stirrup reinforcement in N/mm* which shall not be taken greater than 415 N/mm.

Where the maximum shear stress calculated is less than half the permissible value and in members of minor structural importance such as lintels, this provision need not be complied with.

Introduction to development length

The development length Ld is given by

$$L_d = \varphi \; \sigma_{st} / 4\tau_{bd}$$

where

φ = nominal diameter of the bar,

σ_{st} = stress in bar at the section considered at design load, and

τ_{bd} = design bond stress

The development length includes anchorage values of hooks in tension reinforcement.

For bars of sections other than circular, the development length should be sufficient to develop the stress in the bar by bond.

Anchorage bond

Anchoring Reinforcing Bars

Anchoring bars in tension

A) Deformed bars may be used without end anchorages provided development length requirement is satisfied. Hooks should normally be provided for plain bars in tension.

B) Bends-The anchorage value of bend shall be taken as 4 times the diameter of the bar for each 45^0 bend subject to a maximum of 16 times the diameter of the bar.

C) Hooks-The anchorage value of a standard U-type hook shall be equal to 16 times the diameter of the bar.

flexural bond

At certain locations in a beam high bond stress may arise due to large variation of bending moment over a short distance, that is, high shear force. These bond stresses are called flexural bond stresses and must be checked at the face of a simple support and at the oints of inflection within continous spans. At these locations tensile capacity to be developed is usually small but the rate of change of tensile stress in bars is high.

Concept of Equivalent Shear and Moments

Equivalent shear, *V_e, shall be calculated from the* formula:

$$V_e = V_u + 1.6\, T_u/b$$

Where,

V_e = equivalent shear,

V_u = *shear,*

T_u = torsional moment, and b = *breadth of beam.*

The equivalent nominal shear stress, τ_{ve}, in this case shall be calculated as V_u/bd

substituting *Vu by V,. The values of* τ_{ve} *shall not exceed* the values of $\tau_{c\,max}$, given in Table 20 of IS 456- :2000

Equivalent Moment M_e

Reinforcement for torsion, when required, shall consist of longitudinal and transverse reinforcement.

The longitudinal reinforcement shall be designed to resist an equivalent bending moment, M_e given by

$M_e = M_u + M_t$; where

Mu = bending moment at the cross-section,

$$\boldsymbol{M_t = T_u((1+D/b)/1.7)}$$

Where T_u is the torsional moment, D is the overall depth of the beam and b is the breadth of the beam

UNIT 4

Design of one way and two way slabs

One way and two way slab

Figures 8.18.4a and b explain the share of loads on beams supporting solid slabs along four edges when vertical loads are uniformly distributed. It is evident from the figures that the share of loads on beams in two perpendicular directions depends upon the aspect ratio l^y/l^x of the slab, l^x being the shorter span. For large values of l^y, the triangular area is much less than the trapezoidal area (Fig.8.18.4a). Hence, the share of loads on beams along shorter span will gradually reduce with increasing ratio of l^y/l^x. In such cases, it may be said that the loads are primarily taken by beams along longer span. The deflection profiles of the slab along both directions are also shown in the figure. The deflection profile is found to be constant along the longer span except near the edges for the slab panel of Fig.8.18.4a. These slabs are designated as one-way slabs as they span in one direction (shorter one) only for a large part of the slab when $l^y/l^x > 2$.

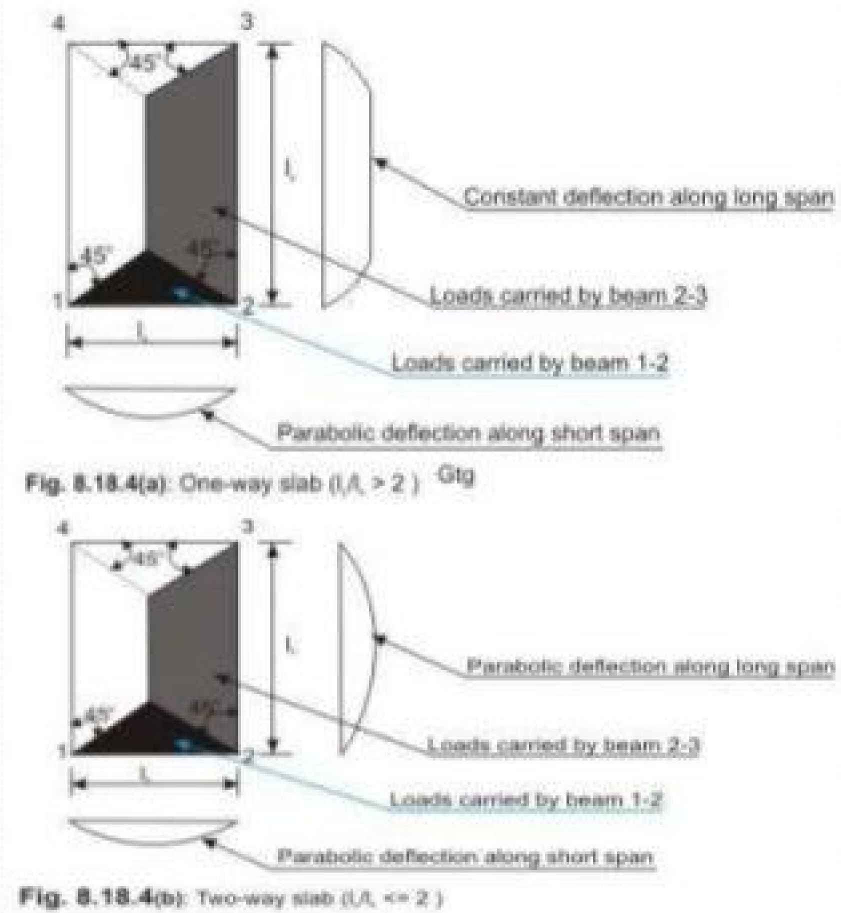

Fig. 8.18.4: Sharing of loads

On the other hand, for square slabs of $l^y/l^x = 1$ and rectangular slabs of l^y/l^x up to 2, the deflection profiles in the two directions are parabolic (Fig.8.18.4b). Thus, they are spanning in two directions and these slabs with l^y/l^x up to 2 are designated as two-way slabs, when supported on all edges

It would be noted that an entirely one-way slab would need lack of support on short edges. Also, even for $l^y/l^x < 2$, absence of supports in two parallel edges will render the slab one-way. In Fig. 8.18.4b, the separating line at 45 degree is tentative serving purpose of design. Actually, this angle is a function of l^y/l^x .

Design Shear Strength of Concrete in Slabs

Experimental tests confirmed that the shear strength of solid slabs up to a depth of 300 mm is comparatively more than those of depth greater than 300 mm. Accordingly, cl.40.2.1.1 of IS 456 stipulates the values of a factor k to be multiplied with $c\tau$ given in Table 19 of IS 456 for different overall depths of slab. Table 8.1 presents the values of k as a ready reference below:

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

Design Consideration

The primary design considerations of both one and two-way slabs are strength and deflection. The depth of the slab and areas of steel reinforcement are to be determined from these two aspects. The detailed procedure of design of one-way slab is taken up in the next section. However, the following aspects are to be decided first

(a) Effective span (cl.22.2 of IS 456)

The effective span of a slab depends on the boundary condition. Table 8.2 gives the guidelines stipulated in cl.22.2 of IS 456 to determine the effective span of a slab.

Sl.No	Support condition	Effective span
1	Simply supported not built integrally with its supports .	Lesser of (i) clear span + effective depth of slab, and (ii) centre to centre of supports
2	Continuous when the width of the support is < $1/12^{th}$ of clear span	Do
3	Continuous when the width of the support is > lesser of $1/12^{th}$ of clear span or 600 mm (i) for end span with one end fixed and the other end continuous or for intermediate spans, (ii) for end span with one end free and other end continuous, (iii) spans with roller or rocker bearings	(i) Clear span between the supports (ii) Lesser of (a) clear span + half the effective depth of slab, and (b) clear span + half the width of the discontinuous support (iii) The distance between the centres of bearings
4	Cantilever slab at the end of a continuous slab	Length up to the centre of support
5	Cantilever span	Length up to the face of the support + half the effective depth
6	Frames	Centre to centre distance

Design contd..

Effective span to effective depth ratio (cls.23.2.1a-e of IS 456)

The deflection of the slab can be kept under control if the ratios of effective span to effective depth of one-way slabs are taken up from the provisions in cl.23.2.1a-e of IS 456. These stipulations are for the beams and are also applicable for one-way slabs as they are designed considering them as beam of unit width.

(c) Nominal cover (cl.26.4 of IS 456)

The nominal cover to be provided depends upon durability and fire resistance requirements. Table 16 and 16A of IS 456 provide the respective values. Appropriate value of the nominal cover is to be provided from these tables for the particular requirement of the structure

Design contd..

(d) Minimum reinforcement (cl.26.5.2.1 of IS 456)

Both for one and two-way slabs, the amount of minimum reinforcement in either direction shall not be less than 0.15 and 0.12 per cents of the total cross-sectional area for mild steel (Fe 250) and high strength deformed bars (Fe 415 and Fe 500)/welded wire fabric, respectively

(e) Maximum diameter of reinforcing bars (cl.26.5.2.2)

The maximum diameter of reinforcing bars of one and two-way slabs shall not exceed one-eighth of the total depth of the slab.

(f) Maximum distance between bars (cl.26.3.3 of IS 456)

The maximum horizontal distance between parallel main reinforcing bars shall be the lesser of (i) three times the effective depth, or (ii) 300 mm. However, the same for secondary/distribution bars for temperature, shrinkage etc. shall be the lesser of (i) five times the effective depth, or (ii) 450 mm.

Design of One-way Slabs

Step 1: Selection of preliminary depth of slab

The depth of the slab shall be assumed from the span to effective depth ratios

Step 2: Design loads, bending moments and shear forces

The total factored (design) loads are to be determined adding the estimated dead load of the slab, load of the floor finish, given or assumed live loads etc. after multiplying each of them with the respective partial safety factors.

Step 3: Determination/checking of the effective and total depths of slabs

The effective depth of the slab shall be determined by

$M^{u,lim} = R^{,lim}\, bd^2$

Design of One-way Slabs contd..

Step 4: Depth of the slab for shear force

Theoretically, the depth of the slab can be checked for shear force if the design shear strength of concrete is known. Since this depends upon the percentage of tensile reinforcement, the design shear strength shall be assumed considering the lowest percentage of steel.

Step 5: Determination of areas of steel

Area of steel reinforcement along the direction of one-way slab should be determined

$M^u = 0.87 f^y A^{st} d \{1 - (A^{st})(f^y)/(f^{ck})(bd)\}$

Step 6: Selection of diameters and spacings of reinforcing bars (cls.26.5.2.2 and 26.3.3 of IS 456)

The diameter and spacing of bars are to be determined as per cls.26.5.2.2 and 26.3.3 of IS 456.

Detailing of Reinforcement

Figures 8.18.5a and b present the plan and section 1-1 of one-way continuous slab showing the different reinforcing bars in the discontinuous and continuous ends (DEP and CEP, respectively) of end panel and continuous end of adjacent panel (CAP). The end panel has three bottom bars B1, B2 and B3 and four top bars T1, T2, T3 and T4. Only three bottom bars B4, B5 and B6 are shown in the adjacent panel. Table 8.3 presents these bars mentioning the respective zone of their placement (DEP/CEP/CAP), direction of the bars (along *x or y) and the resisting moment for which they shall be designed or if to be provided on the basis of minimum reinforcement. These bars are explained below for the three types of ends of the two panels*

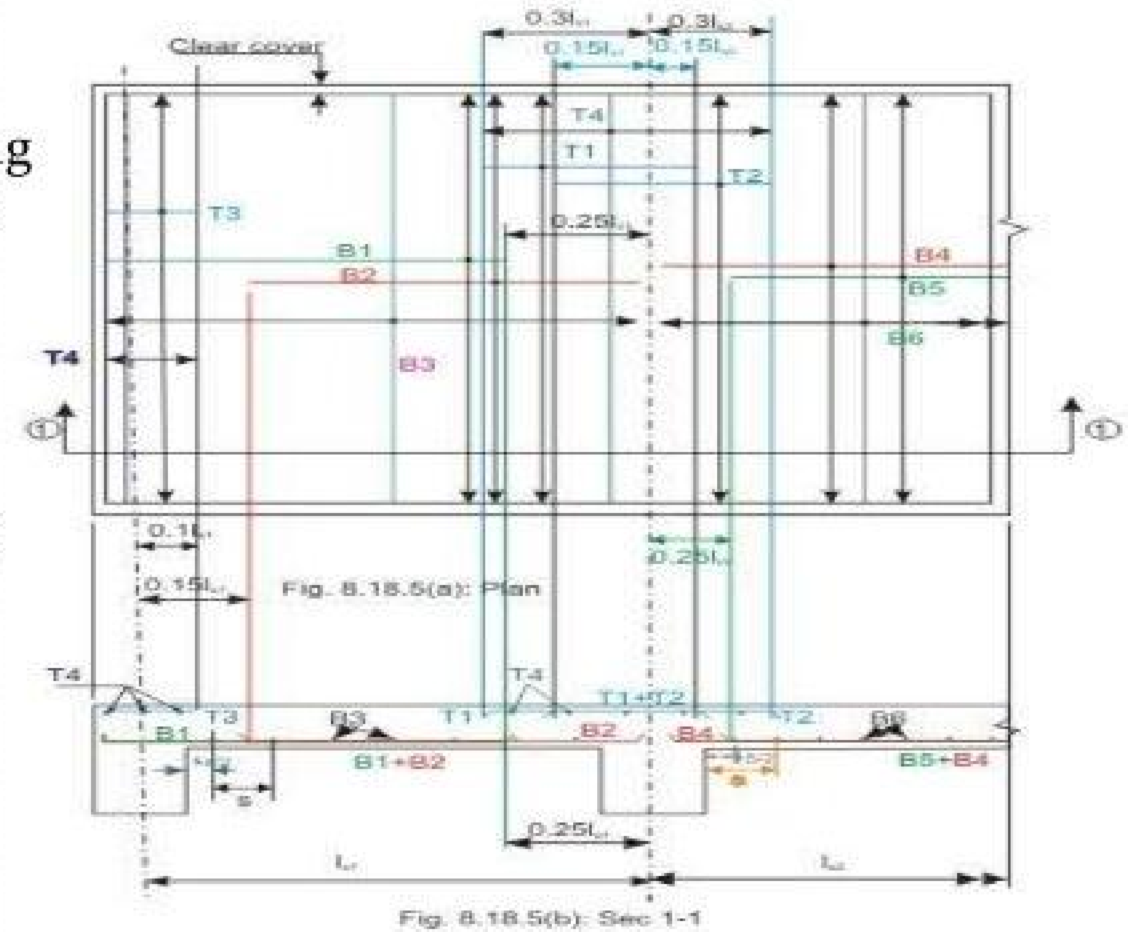

Fig. 8.18.5: Reinforcement of one-way slab

Numerical

Design the one-way continuous slab of Fig.8.18.6 subjected to uniformly distributed imposed loads of 5 kN/m^2 using M 20 and Fe 415. The load of floor finish is 1 kN/m^2. The span dimensions shown in the figure are effective spans. The width of beams at the support = 300 mm.

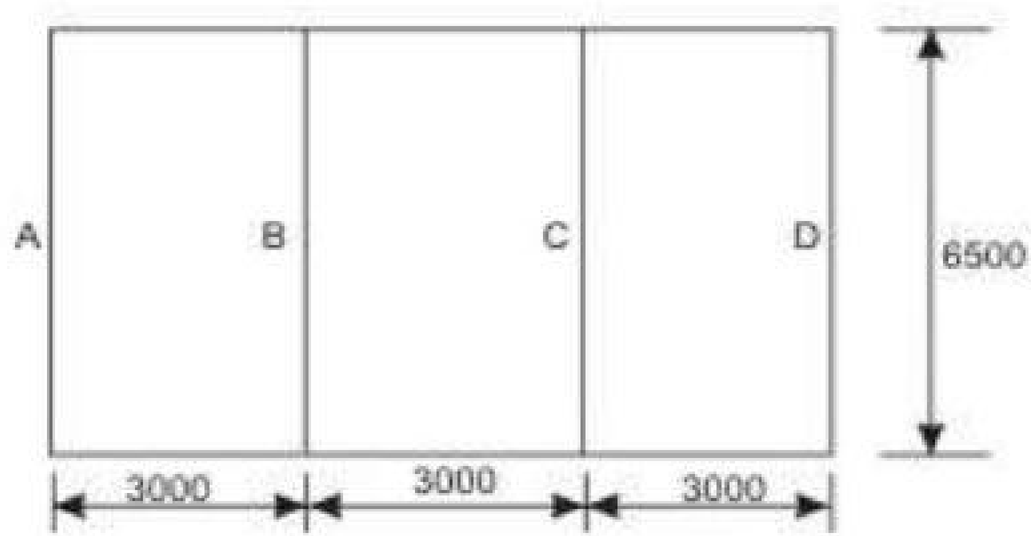

Fig. 8.18.6: Problem 8.1

Solution

Step 1: Selection of preliminary depth of slab

The basic value of span to effective depth ratio for the slab having simple support at the end and continuous at the intermediate is (20+26)/2 = 23 (cl.23.2.1 of IS 456).

Modification factor with assumed $p = 0.5$ *and* $f_s = 240\ N/mm^2$ *is obtained as 1.18 from Fig.4 of IS 456.*

Therefore, the minimum effective depth = 3000/23(1.18) = 110.54 mm. Let us take the effective depth $d = 115$ *mm and with 25 mm cover, the total depth D = 140 mm.*

Step 2: Design loads, bending moment and shear force

Dead loads of slab of 1 m width = 0.14(25) = 3.5 kN/m

Dead load of floor finish =1.0 kN/m

Factored dead load = 1.5(4.5) = 6.75 kN/m

Factored live load = 1.5(5.0) = 7.50 kN/m

Total factored load = 14.25 kN/m

Maximum moments and shear are determined from the coefficients given in Tables 12 and 13 of IS 456.

Maximum positive moment = 14.25(3)(3)/12 = 10.6875 kNm/m

Maximum negative moment = 14.25(3)(3)/10 = 12.825 kNm/m

Maximum shear $V^u = 14.25(3)(0.4) = 17.1\ kN$

Step 3: Determination of effective and total depths of slab

From Eq.3.25 of sec. 3.5.6 of Lesson 5, we have

$M^{u,lim} = R^{,lim} bd^2$ *where* $R^{,lim}$ *is 2.76 N/mm² from Table 3.3 of sec. 3.5.6 of Lesson 5. So,*

$d = \{12.825(10^6)/(2.76)(1000)\}^{0.5} = 68.17$ *mm*

Since, the computed depth is much less than that determined in Step 1, let us keep $D = 140$ *mm and* $d = 115$ *mm.*

Step 4: Depth of slab for shear force

Table 19 of IS 456 gives $c\tau = 0.28$ *N/mm² for the lowest percentage of steel in the slab. Further for the total depth of 140 mm, let us use the coefficient k of cl. 40.2.1.1 of IS 456 as 1.3 to get c ττkc= = 1.3(0.28) = 0.364 N/mm².*

Table 20 of IS 456 gives max$c\tau = 2.8$ *N/mm². For this problem bdVuv/ =τ = 17.1/115 = 0.148 N/mm². Since, maxc cvτττ<<, the effective depth d = 115 mm is acceptable.*

Step 5: Determination of areas of steel

From Eq.3.23 of sec. 3.5.5 of Lesson 5, we have

$M^u = 0.87 f^y A^{st} d \{1 – (A^{st})(f^y)/(f^{ck})(bd)\}$

(i) For the maximum negative bending moment

$12825000 = 0.87(415)(A^{st})(115)\{1 – (A^{st})(415)/(1000)(115)(20)\}$

or - 5542.16 $A2stA^{st} + 1711871.646 = 0$

Solving the quadratic equation, we have the negative $A^{st} = 328.34\ mm^2$

(ii) For the maximum positive bending moment

$10687500 = 0.87(415)\ A^{st}(115)\ \{1 – (A^{st})(415)/(1000)(115)(20)\}$

or - 5542.16 $A2stA^{st} + 1426559.705 = 0$

Solving the quadratic equation, we have the positive $A^{st} = 270.615\ mm^2$

Detailing

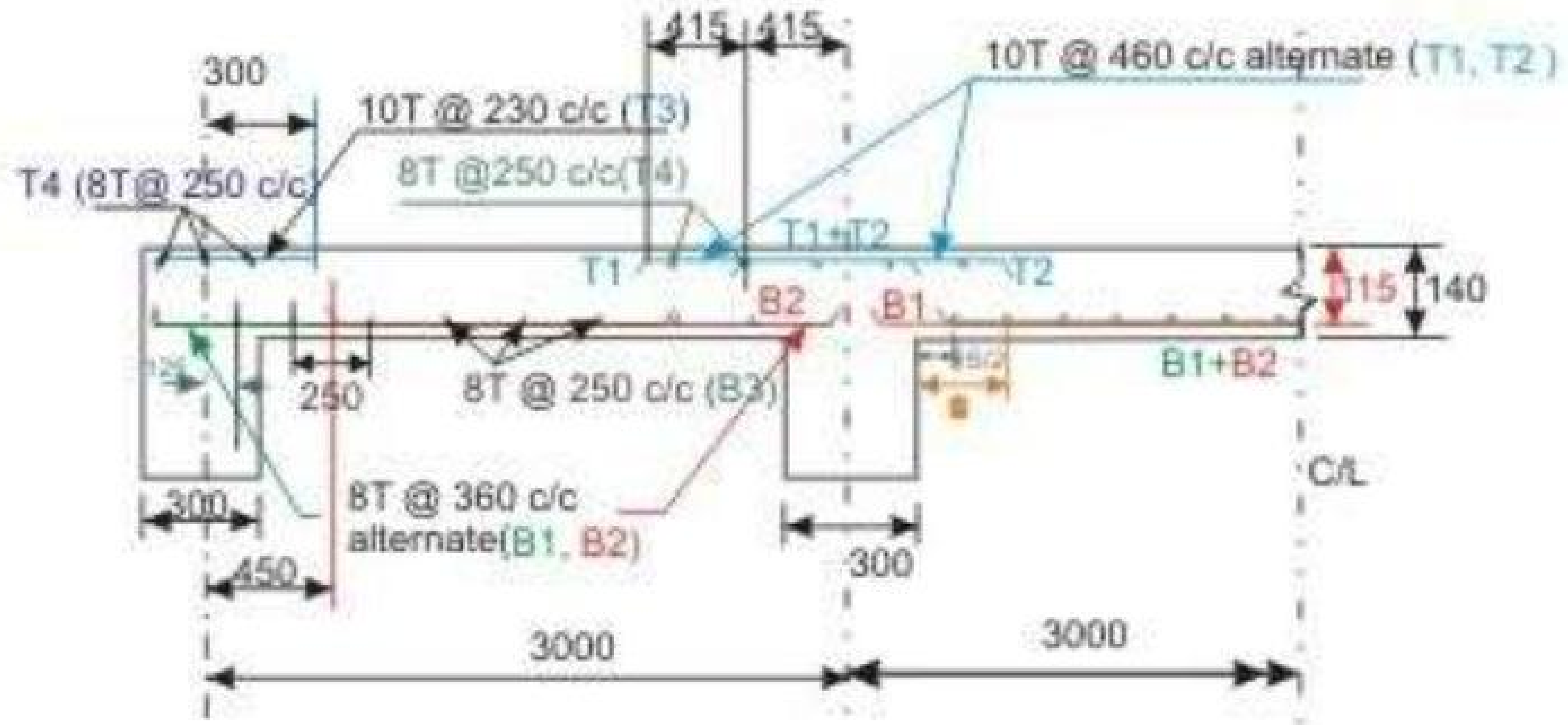

Fig. 8.18.7: Problem 8.1

Serviceability Limit States

In the method of design based on limit state concept, the structure shall be designed to withstand safely all loads liable to act on it throughout its life; it shall also satisfy the serviceability requirements, such as limitations on deflection and cracking.

The structure shall be designed on the basis of the most critical limit state and shall be checked for other limit state

Cracking and Vibrations

Cracking of concrete should not adversely affect appearance or durability of the structure ; the acceptable limits of cracking would vary with the type of structure and environment. Where specific attention is required to limit the designed crack width to a particular value, crack width calculation may be done using formula

The surface width of the cracks should not, in general, exceed 0.3 mm in members where cracking is not harmful and does not have any serious adverse effects upon the preservation of reinforcing steel nor upon the durability of the structures.

UNIT 5
Design of Columns by Limit State Design Method

Effective height of columns

The effective height of the column is defined as the height between the points of contraflexure of the buckled column.The IS 456 2000 has given certain values of the effective height for the normal usuage Assuming idealized end conditions shown in table

Degree of end restraint of the member	Effective Height
Effective held in position and restrained against rotation at both ends	0.65 L
Effectively held in position at both ends, restrained against rotation at one end	0.80 L
Effectively held in position at both ends, but not restrained against rotation	L
Effectively held in position and restrained against rotation at one end, and at the other restrained against rotation but not held in position	1.2 L
Effectively held in position and restrained against rotation in one end, and at the other partially restrained against rotation but not held in position	1.5 L
Effectively held in position at one end but not restrained against rotation, and at the other end restrained against rotation but not held in position	2.0 L

Assumptions

The following are the assumptions for the limit state of collapse in compression in addition with the assumptions of LSM

The maximum compressive strain in concrete in axial compression is taken as **0.002.**

The maximum compressive strain at the highly compressed extreme fibre in concrete subjected to axial compression and bending and when there is no tension on the section shall be 0.0035 minus 0.75 times the strain at the least compressed extreme fibre.

Minimum eccentricity

All columns shall Abe designed for minimum eccentricity, equal to the unsupported length of column/500 plus lateral dimensions/30, subject to a minimum of 20 mm. Where bi-axial bending is considered, it is sufficient to ensure that eccentricity exceeds the minimum about one axis at a time

Where calculated eccentricity is larger, the minimum eccentricity should be ignored.

Short column under axial compression

Factored concentric load applied on short tied columns is resisted by concrete of area Ac and longitudinal steel of areas Asc effectively held by lateral ties at intervals (Fig.10.21.2a of Lesson 21). Assuming the design strengths of concrete and steel are 0.4fck and 0.67fy, respectively, as explained in sec. 10.22.2, we can write

$$P_u = 0.4 f_{ck} A_c + 0.67 f_y A_{sc}$$

where Pu = factored axial load on the member,

fck = characteristic compressive strength of the concrete,

Ac = area of concrete,

fy = characteristic strength of the compression reinforcement,

Asc = area of longitudinal reinforcement for columns.

The above equation, given in cl. 39.3 of IS 456, has two unknowns Ac and Asc to be determined from one equation. The equation is recast in terms of Ag, the gross area of concrete and p, the percentage of compression reinforcement employing

$$Asc = pAg/100$$

$$Ac = Ag(1 - p/100)$$

Accordingly, we can write

$$Pu/Ag = 0.4fck + (p/100)\,(0.67fy - 0.4fck)$$

Column with Helical Reinforcement

Columns with helical reinforcement take more load than that of tied columns due to additional strength of spirals in contributing to the strength of columns. Accordingly, cl. 39.4 recommends a multiplying factor of 1.05 regarding the strength of such columns. The code further recommends that the ratio of volume of helical reinforcement to the volume of core shall not be less than 0.36 (Ag/Ac – 1) (fck/fy), in order to apply the additional strength factor of 1.05 (cl. 39.4.1). Accordingly, the governing equation of the spiral columns may be written as

$$Pu = 1.05(0.4\ fck\ Ac + 0.67\ fy\ Asc)$$

Volume of helical reinforcement in one loop = $\pi(Dc - \varphi sp)$ asp

Volume of core = $(\pi/4)\ Dc^2\ p$

where Dc = diameter of the core

φ sp = diameter of the spiral reinforcement

asp = area of cross-section of spiral reinforcement

p = pitch of spiral reinforcement

To satisfy the condition of cl.39.4.1 of IS 456, we have

Thank You