STRUCTURAL DESIGN LAB

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CERTIFICATE

This is to certify that it is a bonafide record of practical work done in the ……………………………………………………………………………laboratory in ……………………… semester of ....year during the academic year ....by

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INTRODUCTION

ENGINEERING STRUCTURE AND STRUCTURAL DESIGN:

An engineering structure is an assembly of members or elements transferring the load or resisting external actions and providing a form to serve the desired function.

The structural design is a science and art of designing with economy and elegance. A durable structure, which can safely carry the forces and can serve the desired function satisfactorily during its expected service life span.

OBJECT AND BASIC REQUIREMENTS OF STRUCTURAL DESIGN:

- Serviceability
- Safety
- Durability
- Economy
- Aesthetic beauty

SEQUENCE OF STRUCTURAL DESIGN:

1. Design of Slab
2. Design of Beam
3. Design of Column
4. Design of Footing
5. Design of Staircase

DESIGN PROCESS:

Engineering is a professional art of applying the science to the efficient conversion of natural resources for the benefit of man. Engineering, therefore, requires above all creative imagination to innovate useful application for natural phenomenon.

The Design of any structure is categorized into two types:

- Functional Design
- Structural Design

1) *Functional Design*:

The structure to be constructed should primarily serve the basic purpose for which it is to be used and must have a pleasing look.

The following factors comes under Functional Design whether the structure should be a load bearing structure or RCC framed structure or a steel structure, whether the roof shall consist of steel roof trusses and girders or RCC folded plates or a RC beam slab construction, Arrangement of rooms, good ventilation, lighting, acoustics, sufficient head room, proper water supply and drainage etc.

2) *Structural Design*:

Once the form of the structure is selected the structural design process starts.

Structural Design is an art and science of understanding the behaviour of structural members subjected to loads and designing them with economy and elegance to give a safe, serviceable and durable structure.

The principal elements of a RC building frame consists of:

1) Slab to cover a larger area
2) Beams to support slabs and walls
3) Columns to support beams
4) Footing to distribute concentrated column loads over a larger area of the supporting soil such that the bearing capacity of soil is not exceeded.

In a framed structure the load is transferred from slab to beam, from beam to column and then to foundation and soil below it.

STAGES IN STRUCTURAL DESIGN:

The process of structural design involves the following stages:

1) Structural Planning
2) Action of forces and Computation of loads
3) Methods of Analysis
4) Member design
5) Detailing, Drawing and Preparation of schedules.

LOADS AND MATERIALS

INTRODUCTION

Loads and properties of materials constitute the basic parameters affecting the design of RC.structure. Both of them are basically of varying nature. The correct assessment of loads on structure is very important step for the safe and serviceable design of structure

TYPES OF LOADS

The loads are broadly classified as vertical loads, horizontal loads and longitudinal loads. The vertical loads consist of dead load, live load and impact load. The horizontal loads comprise of wind load and earthquake load. The longitudinal loads (tractive and braking forces are considered in special cases of design of bridges, design of gantry girders etc.,)

Dead Loads

Dead Roads are permanent or stationary loads which are transferred to the structure throughout their life span Dead load is primarily due to self-weight of structural members, permanent partition walls, fixed permanent equipment and weights of different materials such as brick stoneetc.

Imposed Loads or Live Loads

Live loads are either movable or moving loads without any acceleration or impact. These are assumed to be produced by the intended use or occupancy of the building including weight of movable partition or furniture etc. The imposed loads to be assumed in buildings are given Table A-2 as per 15:875 (Part-2).The floor slabs have to be designed to carry either uniformly distributed loads and/or concentrated loads whichever produce greater stresses in the part under consideration. Since it is unlikely that anyone particular time all floors will not be simultaneously carrying maximum loading, the code permits some reduction in imposed loads in designing columns, load bearing walls, piers their supports and foundations.

Impact Load

Impact load is caused by vibration or impact or acceleration. A person walking produces a live load but soldiers marching or frames supporting lifts and hoists produce impact loads. Thus, impact load is equal to imposed load increased by some percentage (called impact factor or impact allowance) depending on the intensity of impact.

Wind Load

Wind load is primary horizontal load caused by movement of it relative to earth. The details of design wind load are given in iS: 875 (Part 3) Wind load is required to be considered in design especially when the height of the building exceeds two times the dimensions' transverse to the exposed wind surface. For low rise building say up to 4 to 5 storeys the wind load is not critical because the moment of resistance provided by the continuity of floor system to column connection and walls provided between columns are sufficient to resist the effect of these forces. Further in limit state method, the factor for design load is reduced to 1.2(D L +L.L+W.L) when wind is considered as against the factor of 1.5(DL +L.L) when wind is not considered.

Earthquake Load

Earthquake loads are horizontal loads caused by earthquake and shall be computed in accordance with IS: 1893. For monolithic reinforced concrete structures located in seismic zone II and III with not more than 5 storey high, the seismic forces are not critical (See IS:13920)

CHARACTERISTIC LOAD

Since the loads are variable in nature they are determined based on statistical approach. But it is impossible to give a guarantee that the loads cannot exceed during the life span of the structure. Thus, the characteristic value of the load is obtained based on statistical probabilistic principles from mean value and standard deviation.

DESIGN LOAD

The variation in loads due to unforeseen increase in loads, constructional inaccuracies, type of Limit State etc., are taken into account to define the design load.

The design load is given by:

Design load = γf X characteristic load (clause:36.3.2)

where γf = partial safety for loads given in Table 2.4.1

Table 2.4.1 : Partial Safety Factor (γ_f) for Loads (according to IS : 456-2000)

Load Combination	*Limit State of Collapse*			*Limit State of Serviceability*		
	DL	*IL*	*WL*	*DL*	*IL*	*WL*
DL + IL	1.5	1.5	--	1.0	1.0	--
DL + WL	1.5 *or* 0.9 *	--	1.5	1.0	--	1.0
DL + IL + WL	1.2	1.2	1.2	1.0	0.8	0.8

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

Notes: *(1) DL = dead load , IL = Imposed load or Live load , WL = wind load.*
(2) While considering earthquake effects, substitute EL for WL.
(3) Since the serviceability relates to the behavior of structure at working load the partial safety factors for limit state of serviceability are unity.
(4) For limit state of serviceability, the values 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 load likely to be permanent may only be considered. For details see " Limit State Theory and Design of Reinforced Concrete[26] " Chapter . 8

EXERCISE I

ANALYSIS AND DESIGN OF PLANE FRAME

The roof of an 8 m wide hall is supported on a portal frame spaced at 4 m intervals. The height of the portal frame is 4 m. The continuous slab is 120 mm thick. Live load on roof = 1.5 kN/m^2. Bearing capacity of the soil is 150 kN/m^2. The columns are connected with a plinth beam and the base of the column may be assumed as fixed. Design the column and beam members and a suitable foundation footing for the columns of the portal frame.

Adopt M-20 grade concrete and Fe-415 HYSD bars.

1. *Data*

Spacing of portal frame = 4 m

Span of portal frame = 8 m

Height of columns = 4 m

Live load on roof = 1.5 kN/m^2

Materials: M20 grade concrete and Fe415 HYSD bars.

2. *Characteristic strength*

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

3. *Design of slab*

Provide 120 mm slab with 10 mm bars at 200 mm centres at supports and mid span sections.

Distribution steel is 6 mm Φ at 130 mm centres.

4. *Design of portal frame*

Effective span of beam = 8 m

$$Effective\ depth = d = \left(\frac{8000}{12}\right) = 666\ mm$$

Adopt d = 650 mm arid overall depth = 700 mm, breadth = b = 400 mm. Column section is assumed as 400 mm by 600 mm.

(a) load on frame

Self-weight of slab = (0.12 x 24)	= 2.88 kN/m^2
Roof finish	= 0.50
Ceiling finish	= 0.25
Live load	= 1.50
Total load	= 5.13 kN/m^2
Load from slab= (5.13 x 4)	= 20.52 kN/m
Self-weight of beam= (0.4 x 0.58 x 24)	= 5.56
Finishes of beam	= 0.92
Total load on beam	= 27.00 kN/m

The moments in the portal frame fixed at base and loaded as shown in Fig. are analysed by moment distribution.

AB = 4 m, BC = 8 m

$$I_{AB} = \left(\frac{400 \times 600^3}{12}\right) \qquad I_{BC} = \left(\frac{400 \times 700^3}{12}\right)$$

$$I_{AB} : I_{BC} = 1 : \left(\frac{700}{600}\right)^3 = 1 : 1.57$$

(b) Relative stiffness

$$k_{AB} = \left(\frac{I}{4}\right) = 0.25I$$

$$k_{BC} = \left(\frac{1.57I}{8}\right) = 0.20I$$

(c) Distribution factor

$$d_{BA} = \left(\frac{0.25I}{0.25 + 0.20I}\right) = 0.55$$

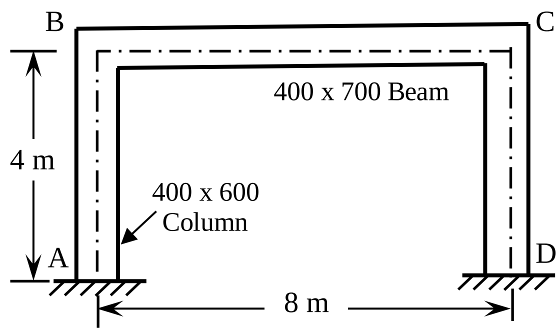
$$d_{BC} = \left(\frac{0.20I}{0.25 + 0.20I}\right) = 0.45$$

(d) Fixed end moments

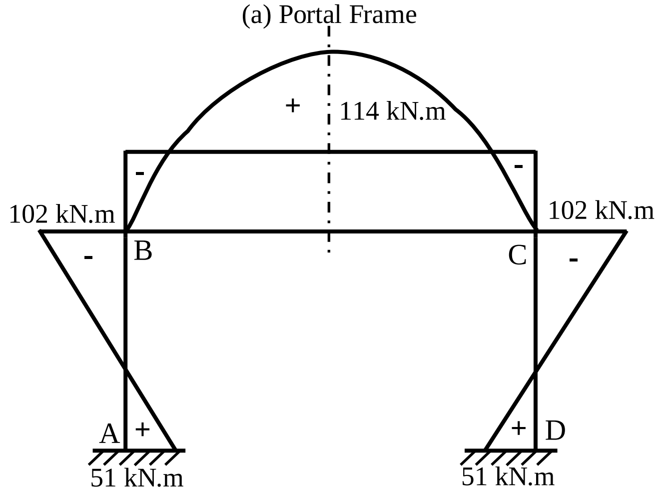
$$F_{BC} = -\left(\frac{wL^2}{12}\right) = -\left(\frac{27 \times 8^2}{12}\right) = -144\ kN.m$$

$$F_{CB} = +\left(\frac{wL^2}{12}\right) = +\left(\frac{27 \times 8^2}{12}\right) = +144\ kN.m$$

$$F_{BA} = F_{AB} = 0$$



(a) Portal Frame



(b) Bending Moment Diagram

(e) Moment distribution

	B		C		
A	**0.55**	**0.45**	**0.45**	**0.55**	**D**
0	0	-144	+144	0	0
39.5	+79	+65	-65	-79	-39.5
		-32.5	32.5		
8.9	17.8	14.7	-14.7	-17.8	-8.9
		-7.4	7.4		
2.0	4.0	3.4	-3.4	-4.0	-2.0
		-1.7	1.7		
0.47	0.93	0.77	-0.77	-0.93	-0.47
51	**+102**	**-102**	**-102**	**-102**	**-51**

(f) Design moments and shear forces

Service load moments: M_B = 102 kN.m and M_A= 51 kN.m

Design ultimate moments are computed as:

M_{uB} = (1.5 x 102) = 153 kN.m

M_{uA} = (1.5 X 51) = 76.5 kN.m

Maximum positive service load moment at centre of span BC is:

= [0.125 wL^2 - 102]

= [(0.125 x 27 x 82) -102] = 114 kN.m

Ultimate moment at centre of span BC = M_u = (1.5 x 114) = 171 kN.m

The service load bending moment diagram for the portal frame is shown in Fig.

Maximum working shear force at B = (0.5 x 27 x 8) = 108 kN

Maximum ultimate shear force at B = V_u = (1.5 x 108) = 162 kN

$$\text{Working load shear force at } A = \left(\frac{M_A + M_B}{4}\right) = \left(\frac{51 + 102}{4}\right) = 38.25\ kN$$

Ultimate shear force at A = (1.5 x 38.25) = 57.37 kN

(g) Design of beam section

Mid span section is designed as a tee section.

Factored design moment= M_u= 171 kN.m

Effective width of Flange *(b_f)* is given by the relation:

$$b_f = \left[(L/6) + b_w + 6D_f\right]$$

$$b_f = [(8/6) + 0.4 + (6 \times 0.12)] = 2.45\ m < 4m$$

Hence, b_f= 2450 mm and effective depth = d = 650 mm

x_u = 0.48 d = (0.48 x 650) = 312 mm

$$\left[\frac{D_f}{d}\right] = \left[\frac{120}{650}\right] = 0.18 < 0.2$$

$$\left[\frac{D_f}{x_u}\right] = \left[\frac{120}{320}\right] = 0.38 < 0.43$$

Since the moment value is small, determine the value of neutral axis depth, x_u falling within the flange using the equation:

$$M_u = 0.36 f_{ck} x_u b_f (d - 0.42 x_u)$$

$$(171 \times 10^6) = [(0.36 \times 20 \times x_u \times 2450)(650 - 0.42 \times x_u)]$$

Solving, we have x_u = 15 mm < D_f = 120 mm

Hence considering the section as rectangular, the area of reinforcement required is computed by the reaction:

$$\therefore M_u = 0.87 f_y A_{st} d \left[1 - \frac{A_{st}.f_y}{b d f_{ck}}\right]$$

$$(171 \times 10^6) = (0.87 \times 415 A_{st} \times 650)\left[1 - \frac{415\ A_{st}}{(2450 \times 650 \times 20)}\right]$$

Solving A_{st} = 973 mm^2

Provide 4 bars of 20 mm diameter (A_{st} = 1256 mm^2)

Support section

Factored design moment= M_u = 153 kN.m

The support section is designed as rectangular section and the reinforcement area 18 computed using the equation:

$$(153 \times 10^6) = (0.87 \times 415 A_{st} \times 650)\left[1 - \frac{415\, A_{st}}{(400 \times 650 \times 20)}\right]$$

Solving A_{st}= 687 mm^2 >$A_{st.min}$= 512 mm^2

Since the section has to resist shear force also, provide 4 bars of 20 mm diameter providing an area of 1256 mm^2.

$$\text{Nominal shear stress} = \tau_v = \left(\frac{V_u}{bd}\right) = \left(\frac{162 \times 10^3}{400 \times 650}\right) = 0.62\ N/mm^2$$

$$p_t = \left(\frac{100 A_{st}}{bd}\right) = \left(\frac{100 \times 1256}{400 \times 650}\right) = 0.42$$

Refer Table-19 (IS: 456-2000) and read out the permissible shear stress as:

$$\tau_c = 0.47\ N/mm^2 < \tau_v$$

Hence shear reinforcements are required.

Balance shear force = V_{us} = [162 - (0.47 x 400 x 650) 10^{-3}] = 40 kN

Using 6 mm diameter two-legged stirrups, spacing is given by the relation:

$$s_v = \left[\frac{0.87 f_v A_{sv} d}{V_{us}}\right] = \left[\frac{0.87 \times 415 \times 2 \times 28 \times 650}{40 \times 10^3}\right] = 328\ mm$$

Adopt 300 mm spacing near the supports, gradually increasing to 400 mm towards the centre of span.

(h) Design of column section

Section at top (B)

Moment M = 102 kN.m

Thrust P = 108 kN

Using a load factor of 1.5

M_u= (102 *x* 1.5) = 153 kN.m

P_u= (108 *X* 21.5) = 162 kN

Column section is of size 400 x 600 mm

$\therefore b = 400\ mm$ $\qquad d = 550\ mm$

$D = 600\ mm$ $\qquad d' = 50\ mm$ $\qquad \left(\frac{d'}{D}\right) = 0.10$

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

$$\left(\frac{M_u}{f_{ck}.b.D^2}\right) = \left(\frac{153 \times 10^6}{20 \times 400 \times 600^2}\right) = 0.053$$

$$\left(\frac{P_u}{f_{ck}.b.D}\right) = \left(\frac{160 \times 10^3}{20 \times 400 \times 600}\right) = 0.033$$

Using interaction curves of SP-16 we get:

$$\left(\frac{p}{f_{ck}}\right) = 0.03 \qquad \therefore p = (20 \times 0.03) = 0.6$$

$$A_s = (p.b.D/100) = (0.6 \times 400 \times 600)/100 = 1440\ mm^2$$

$$But\ minimum\ area\ of\ steel = \left(\frac{0.8 \times 400 \times 600}{100}\right) = 1920\ mm^2$$

Provide 4 bars of 20 mm Φ on each face (A_{st} = 2512 mm^2) and 8 mm Φ ties at 300 mm centres throughout the column.

Same reinforcements are provided at the base section of the column.

(i) Design of foundations

Factored design moment= (1.5 x 25.6) = 38.4 kN.m

Effective depth required for footing slab is computed as:

$$d = \sqrt{\frac{M_u}{0.138 f_{ck} b}} = \sqrt{\frac{38.4 \times 10^6}{0.138 \times 20 \times 1000}} = 118\ mm$$

Depth required to resist shear force is greater than that obtained from moment considerations. Hence adopt overall depth of 300 mm

Effective depth = *d* = 250 mm

Area of reinforcements are obtained by computing the parameter:

$$\left(\frac{M_u}{bd^2}\right) = \left(\frac{38.4 \times 10^6}{1000 \times 250^2}\right) = 0.614$$

Refer Table-2 of SP: 16 (Design Tables) and read out the percentage steel as:

$$p_t = \left(\frac{100 A_{st}}{bd}\right) = 0.178$$

$$A_{st} = \left(\frac{0.178 \times 1000 \times 250}{100}\right) = 445\ mm^2 > A_{st,min}$$

Provide 12 mm diameter bars at 150 mm centers both ways (A_{st} = 754 mm^2). Check for shear.

Shear force acting at a distance of 250 mm from the face of the column is given by:

$$Working\ shear\ force = V = \left(\frac{103 + 79}{2}\right) 0.5 = 45.5\ kN$$

Ultimate shear force = V_u = (1.5 x 45.5) = 68.25 kN

$$\tau_v = \left(\frac{68.25 \times 10^3}{1000 \times 250}\right) = 0.273$$

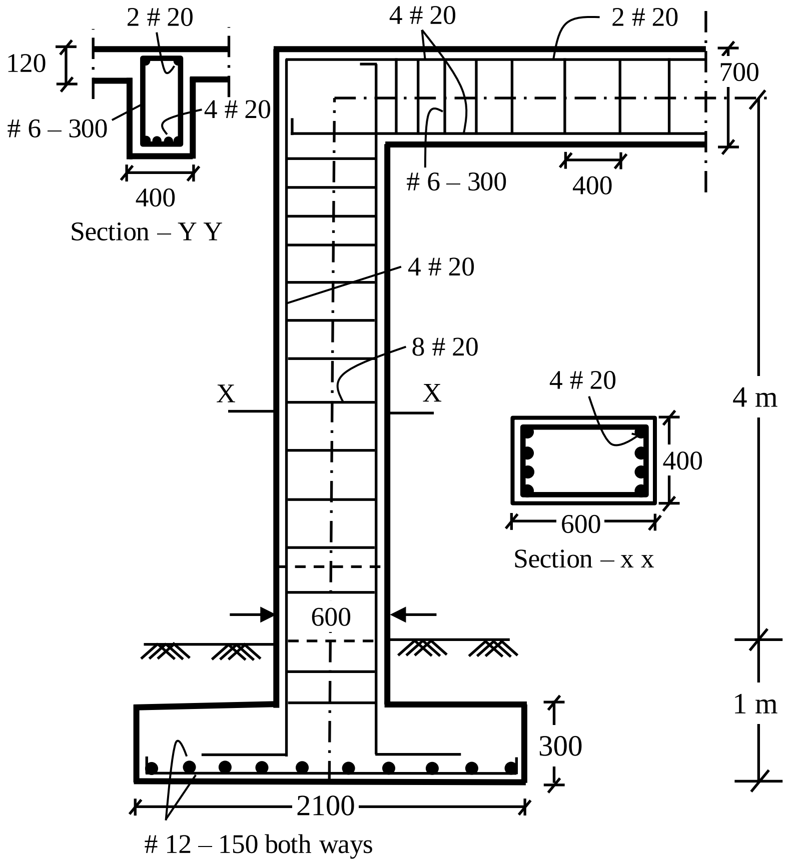
$$\left(\frac{100A_{st}}{bd}\right) = \left(\frac{100 \times 754}{1000 \times 250}\right) = 0.30$$

Refer Table-19 of IS: 456-2000 and read out the design shear strength of concrete τ_c as:

$$\tau_c = 0.38\ N/mm^2$$

Since $\tau_c > \tau_v$, the footing slab safely resist the shear stresses.

The details of reinforcements in the portal frame are shown in Fig.



Reinforcement details in portal frame

EXERCISE-II

DESIGN OF BEAMS

A reinforced concrete beam should be able to resist tensile, compressive and shear stresses induced in it by the loads on the beam. Concrete is fairly strong in compression but very weak in tension. Plain concrete beams are thus limited in carrying capacity by the low tensile strength. Steel is very strong in tension. Thus, the tensile weakness of concrete is overcome by the provision of reinforcing steel in the tension zone around the concrete to make a reinforced concrete beam.

- **There are two types of beams.**

  A. Singly reinforced beams.

  B. Doubly reinforced beams

A) Singly reinforced beams:

In singly reinforced simply supported beams reinforcing steel bars are placed near the bottom of the beam where they are most effective in resisting the tensile bending stresses. In singly reinforced cantilever beams reinforcing bars are placed near the top of beam.

B) Doubly reinforced beams:

A doubly reinforced beam is reinforced both in compression and tension regions. The section of the beam may be a rectangular, T or L section. The necessity of using steel in the compression zone arises due to two main reasons as follows

- When the depth of the beam is restricted the strength available from a singly reinforced beamis inadequate.
- At the support of continuous beams where bending moment changes sign.

<u>IS CODE PROVISIONS</u>:

1) The load on the beam is taken as per clause 24.5 of IS: 456-2000.
2) For continuous beam with equal/unequal spans and equal/unequal loaded, the bending moment is obtained by using Kani's method.
3) Effective span and effective depth of beam is same as explained in slab provisions.
4) The beams at mid span are designed as T-beams and the same steel reinforcement is provided

for all beams and the reinforcement provided is minimum.

5) At supports when the moment of resistance exceeds the balancing moment, the section is designed as double reinforced section.

6) Minimum reinforcement in the tension shall not be less than $\frac{Ast}{bd} = \frac{0.85}{f_y}$

=› Clause 26.5.1.1(a).

7) Maximum reinforcement in tension shall not be exceeded by 0.04bD

=› Clause 26.5.1.1(b).

8) Maximum area of compression reinforcement shall not exceed 0.04bD and reinforcement is enclosed by strength vide =› Clause 26.5.1.2.

9) Nominal shear stress for uniform depth shall be calculated from the equation

$$\tau_v = \frac{v_u}{bd}$$ => Clause 40.1

10) Minimum shear reinforcement will be provided when $\tau_v < \tau_c$ given in table 19.

11) Maximum spacing of shear reinforcement shall not exceed the least of 0.75d or 300 mm for vertical stirrups vide =› clause 26.5.1.5

12) Shear reinforcement shall be provided to carry a shear equal to $v_u - bd.$ The strength of the shear reinforcement v_s shall be calculated for vertical stirrups.

$$v_s = \frac{0.87 f_y A_{sv} d}{S_v}$$ =› clause 40.4(a)

13) At least 1/3rd positive moment reinforcement in simple beam and 1/4th positive moment reinforcement in continuous beam shall extend along the same face of the member into the support to a length equal to $\frac{L_d}{3}$ =› clause 26.2.3.

14) The minimum shear reinforcement in the form of stirrups shall be provided such that

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

=› clause 26.5.1.6

Where, $\mathbf{A_{sv}}$ = Total cross-sectional area of stirrup legs effective in shear

$\mathbf{S_v}$ = Stirrup spacing along the length of the member

b = Breadth of the beam or breadth of flanged beam

f_v = Characteristic strength of the stirrup reinforcement in N/mm^2 which shall not be taken greater than 415N/mm^2.

15) Clear cover for longitudinal reinforcement in a beam, neither less than 25mm nor less than dia of such bar and 15mm to stirrups.
16) At each end of reinforcing bar neither less than 25mm nor less than twice dia of such bar.
17) At least two bars should be used as tension steel, and not more than 6 bars should be used in one layer of beam.
18) The diameter of hanger bars shall not be less than 10mm, and of main tension bars 12 mm. The usual diameter of bar chosen for beams are 10, 12, 16, 20, 22, 25, and 32mm. When using different sized bars in one layer place the largest diameter bars near the faces, there as of steel should be symmetrical about centre line of column as far as possible.
19) The minimum distance between bars has the diameter of bar or maximum size of Aggregate plus 5 mm. Size of aggregate normally used in India is 20 mm. So that clear max distance between bars should be 25 mm.
20) The depth of beam should satisfy the deflection requirements with respect to L/d ratios. In addition, for economy, the ratio of overall depth to which should be between 1.5 to 2.0.
21) Specifications Regarding Spacing Or Stirrups In Doubly Reinforced Beams:

Compression steel placed in doubly reinforced beams also has to be restrained against local buckling during its action like the compression steel. The same rules regarding restraining of column reinforcements by lateral ties apply to compression reinforcements in beams also.

Accordingly, the diameter of stirrups (ties) should be 6mm and the pitch should not be more than the least of the following:

a) Least lateral dimension
b) 16× dia of longitudinal bar
c) 300 mm.

22) Minimum steel is necessary to

a) Guard against any sudden failure of a beam if concrete cover burst and the bond to the tension steel is lost.
b) Prevent brittle failure, which can occur without shear steel.

DESIGN OF CONTINUOUS BEAM

Design a continuous rectangular span 6m to carry a DL of 15kN/m and LL of 20kN/m. the beam is continuous over more than 3 spans. Use M20 concrete and Fe – 415 steels.
Solution

1. Data

Span 6m
DL = 15kN/m
LL = 20kN/m
M20 concrete and Fe – 415 steels

2. Effective Depth

Effective depth may be assumed between $\frac{l}{15}\ and\ \frac{l}{20}$.
Let effective depth be $\frac{l}{15} = \frac{6000}{15}$ = 400mm.
Assume 50mm as effective cover.
D = 400 + 50 =450mm.

Let us assume width of beam = 230mm.
Effective size of beam = 230 * 400 mm^2.

3. Loadings

Self-weight of beam = ɤbD =25*0.23*0.45
= 2.59kN/m.

DL = 15kN/m.
Total DL = 2.59 + 15 = 17.59kN/m.
Factored Load:

DL =1.5*17.59 = 26.38kN/m.
LL =1.5*20 =30kN/m.

4. Maximum Moments and Shear Force

FROM TABLE 12, 13 WE GET BENDING MOMENT COEFFICIENTS AND SHEAR COEFFICIENTS AT SUPPORTS AND SPANS.

Maximum BM (negative) at B and C:

$$Mu = -\left(\frac{1}{10} wl^2 + \frac{1}{9} wl^2\right)$$
$$= -\left(\frac{1}{10} * 26.38 * 6^2 + \frac{1}{9} * 30 * 6^2\right)$$
$$= -214.97 \text{kN-m.}$$

Maximum BM (positive) occurs at middle of end span AB or CD.

Mu $= (\frac{1}{12}wl^2 + \frac{1}{10}wl^2)$
$= (\frac{1}{12} * 26.38 * 6^2 + \frac{1}{10} * 30 * 6^2)$
= 187.14kN-m.

Maximum SF occurs at support next to the end support at B and C.

Vu = (0.6*wl) + (0.6*wl)
= (0.6*26.38*6) + (0.6*30*6)
= 202.9725kN.

5. Design for flexure

Mu, lim = 0.138*fck*b*d^2
= 0.138*20*230*400^2
= 101.57kN-m.

- At mid span of AB or CD**,** Mu = 187.14 kN-m
- Since, Mu > Mu, lim. Design the beam as “Doubly reinforced concrete beam”

Mu_2 = Mu – Mu, lim
= 187.14 – 101.57
= 85.57kN-m

To find f_{sc} compare $\frac{d'}{d}$ value in SP-16 charts, table F, pg no. 13.

$f_{sc}(\frac{N}{mm^2})$				
Grade of steel	$\frac{d'}{d}$			
	0.05	0.10	0.12	0.20
250	217	217	217	217
415	355	353	342	329
500	458	412	395	370
550	458	441	419	380

Fsc = 342N/mm^2.

To find Asc

$M_{u,\,lim}$ = fsc*Asc*(d-d')

Asc $= \frac{342*(400-50)}{85.57*10^6}$
$= 714.87mm^2$.

To find Ast1

For M20 & Fe-415 $P_{t,\,lim}$ = 0.96% (from SP-16, pg no.10, table -E)

Ast1 $= P_{t,\,lim}*b*d$

$= \frac{0.96}{100} * 230 * 400$
$=883.2mm^2$

To find Ast2

Ast2 $= \frac{fsc*Asc}{0.87*fy}$

$= \frac{342*714.87}{0.87*415}$

$= 667.15mm^2$.

Ast = Ast1 + Ast2.
$= 1560.35mm^2$

Assume dia of bars as 20mm at tension zone.
No of bars $= \frac{1560.35}{\frac{\pi}{4}*20^2} \approx 5$.

Assume dia of bars as 20mm at compression zone.
No of bars $= \frac{714.87}{\frac{\pi}{4}*20^2} \approx 3$.

At support B or C

Mu = -214.97kN-m.

Since Mu > $M_{u,\,lim}$, design the beam as "Doubly reinforced concrete beam".

Muz = Mu – Mu, lim = 214.97 – 101.57 = 113.4kN-m.

To find Asc

To find fsc compare $\frac{d'}{d}$ value in SP-16 charts, table F, pg no. 13.

Fsc $= 342N/mm^2$

$M_{u,lim}$ = fsc*Asc*(d-d')

Asc = $\frac{342*(400-50)}{113.4*10^6}$ = 947.37mm^2.

To find Ast1

For M20, Fe-415 Pt, lim = 0.96% (from SP-16, pg no.10, table -E)

Ast1 = Pt, lim*b*d

= $\frac{0.96}{100} * 230 * 400$

=883.2mm^2.

To find Ast2

Ast2 = $\frac{fsc*Asc}{0.87*fy}$

= $\frac{342*947.37}{0.87*415}$

= 897.38mm^2.

Ast =Ast1 + Ast2

= 1780.58mm^2.

Assume dia of bars as 20mm at tension zone.

No of bars = $\frac{1780.58}{\frac{\pi}{4}*20^2}$ ≈ 6.

Extend the tension bars up to $\frac{l}{4}$ on either side from centre.

Assume dia of bars as 20mm at compression zone → No of bars = $\frac{947.37}{\frac{\pi}{4}*20^2}$ ≈ 3.

6. Check for shear

τ_v = $\frac{Vu}{b.d}$

= $\frac{202.97*10^3}{230*500}$

= 0.96%.

% Pt = 0.96

τ_c = 0.6N/mm^2 from IS 456: 2000, pg. no 73

$\tau_{c,max}$ = 2.8N/mm2.

$\tau_c > \tau_v$, design for shear reinforcement.

V_{us} = $V_u - \tau_c$.b.d

= 202.97*10^3 – 0.6*230*400

= 147772.5kN.

Provide 2 legged 8mm dia stirrups.

$$Sv = \frac{0.87 * fy * Asv * d}{Vus}$$

$$= \frac{0.87 * 415 * 2 * \frac{\pi}{4} * 8^2 * 400}{147772.5}$$

$$= 98.20$$

Provide 100mm centre to centre spacing.

7. Check for deflection

$L/d_{provided} < L/d_{allowed}$

$L/d_{allowed}$ = basic value * Modification factor.
= 26*1 = 26 (from IS 456:2000, pg. 37,38)

L/d, provided = (6*1000)/400 = 13.33

Provided < allowed "SAFE"

EXERCISE-III

DESIGN OF COLUMNS

LOADING ON A COLUMN:

The load on columns is calculated by trapezoidal method. This method is used when the loadson the beams coming from the slabs and walls are known prior to column design. The load on column at each floor level is given by,

P_u = Half of the loads coming from the beams (resting on that column) + P_{self} (factored)

P_{self} = Self weight of the column at the floor level under consideration.

IS CODE PROVISIONS FOR DESIGN OF COLUMNS

1. When the ratio of effective length to its least lateral dimension does not exceed 12 i.e., L_e / D <12, then the column is said to be short column. If L_e / D < 12, the column is a long column. Load on long column = C_r × Load on short column, where C_r = [1.25-(L_e / 48 b)]
2. All columns shall be designed for minimum eccentricity subjected to a minimum of 20 mm (clause25.4)
3. The percentage of longitudinal steel should not be less than 0.8, not more than 6% of gross cross- sectional area of the column. With very little steel, Shrinkage effects will cause high stress in steel. With large amount of steel, there will prevent proper placement of concrete, since usually lapping of all column bars is done at the base of each column height so that the percentage of steel at the lapping point will double the percentage at the centre of the column. Hence, even though a max of 6% is allowed, normally the percentage of steel in columns should not exceed 6%
4. In columns of cross-sections larger than those required to carry the load, these minimum areas ofsteel are to be based on the concrete area required to resist the direct stress, and not on the actualarea of concrete in the column.
5. The minimum number of longitudinal bars provided in a column shall be 4 in a rectangular columnand 6 in a circular column.
6. The bars shall not be less than 12 mm in diameter.
7. Spacing of longitudinal bars measured along the periphery of the column shall not exceed 30cm.

8. The pitch of the transverse reinforcement shall not be more than the least of the following :

- The least lateral dimension of the member.
- 6 times the smallest diameter of the longitudinal reinforcement bar.
- 300 mm diameter of tie shall not be less than 1/4th of the diameter of largest longitudinal bar and in no case less than 5 mm.

9. Unsupported length of columns is taken as per clause 24.1.3(b) and the Effectivelength is taken from table 24 of IS: 456-2000.
10. Columns are to be checked in limit state method using chart given in specification 16 IS: 456-2000.
11. The cover to the longitudinal reinforcement bar in a column not less than 40 mm or less thandiameter of such bar in case of column of minimum dimension of 200 mm or under where reinforcing bars do not exceed 12 mm a cover of 25 mm may be used.

COLUMN DESIGN

DESIGN PROBLEM ON BI-AXIAL BENDING:

Design of the reinforcements in a short column 400mm X 600mm, subjected to an ultimate axial load of 1600kN together with ultimate moments of 120kN-m and 90kN-m about the major and minor axes respectively .Adopt M20 Grade of concrete and Fe 415 grade of steel. Take effective cover as 60mm.

1. Data

b = 400mm	M_{ux}= 120 kN-m
D= 600 mm	M_{uy} = 90kN-m
P_u= 1600 kN	f_{ck} = 20 N/mm^2
d^1= 60 mm	f_y= 415 N/mm^2

2. For M_{ux1} : Along the X-direction

D = 600 mm b= 400mm

Assume 1% of steel reinforcement as longitudinal bars

A_{sc} = 1% A_g

= 0.01 * A_g

A_{sc} = 0.01 x 400 x 600

A_{sc}= 2400 mm^2

Assume, 16mm of the bars of longitudinal bars

$A_{sc\ single}$ $= \pi/4 \times 16^2$ $= 201.06\ mm^2$

Number of bars (n) $= \frac{2400}{201.06} = 11.93 \sim 12$

$A_{sc\ provided}$ $= 12 \times \pi/4 \times 16^2$

$= 2412.74\ mm^2$

Percentage of steel (p) $= 100\ X\ \frac{Asc}{bD}$

$= \frac{100\ X\ 2412.74}{400\ \ X\ 600}$

$= 1.005\%$

Now $\frac{P}{f_{ck}} = \frac{1.005}{20} = 0.05$

$$\frac{P_u}{f_{ck}bD} = \frac{1600\ X\ 10^3}{20\ X\ 400\ X\ 600} = 0.33$$

And, $\frac{d'}{D} = \frac{60}{600} = 0.1$

Let us assume the reinforcement provided about both directions

As the value of $\frac{d'}{D} = 0.1\ ,\frac{P_u}{f_{ck}bD} = 0.33, \frac{P}{f_{ck}} = 0.05$

And, Fe 415 , so take chart No :44 , of Sp-16

$\frac{M_{ux1}}{f_{ck}bD^2} = 0.09$ $\rightarrow$ M_{ux1}=0.09 x 20 x 400 x 600^2

$M_{ux1} = 254.2\ kNm$

3. For M_{uy1}: Along y-Direction:

Here b = 600 mm & D = 400 mm

$\frac{P}{f_{ck}} = 0.05$ $\frac{P_u}{f_{ck}\ bD} = 0.33$ $\frac{d'}{D} = \frac{60}{400} = 0.15$

As the value of $\frac{d'}{D} = 0.15$ $\frac{P}{f_{ck}} = 0.05$ $\frac{P_u}{f_{ck}\ bD} = 0.33$, Fe = 415

so take chart no: 45, of Sp − 16 $\rightarrow$ $\frac{M_{uy1}}{f_{ck}bD^2} = 0.08$

$M_{uy} = 0.08 \times 20 \times 600 \times 400^2$
$M_{uy1} = 153.66$ kN-m

4. Biaxial Condition As per IS 456:2000

$$If \quad \left[\frac{M_{ux}}{M_{ux1}}\right]^{\alpha_n} + \left[\frac{M_{uy}}{M_{uy1}}\right]^{\alpha_n} \leq \; 1 \qquad \rightarrow \text{OK}$$

The designed column is safe under being

Now, $P_{UZ} \quad = 0.45\, f_{ck} A_c + 0.75\, f_y A_{sc}$

$=0.45 \times 20 \times (400 \times 600 - 2412.74) + 0.7 \times 415 \times 2412.74$

$P_{uZ} \quad = 2889.25$kN

Now $\quad \frac{P_u}{P_{U7}} = \frac{1600}{2889.25} = 0.55$

As per IS : 456 : 2000 Pg no 71

$\frac{P_u}{P_{uZ}}$	α_n
≤ 0.2	1
≥ 0.8	2

$\alpha_n = 1 + \left(\frac{2-1}{0.8-0.7}\right) x\; (0.55 - 0.2)$
$\alpha_n = 1.58$

Now, $\left[\frac{120}{259.2}\right]^{1.58} + \left[\frac{90}{153.66}\right]^{1.58} = 0.72 \quad < 1$

Hence, the designed column is safe under bi-axial loading

5. Design of Lateral Ties

I. Diameter of 20mm
i>. $\nless \frac{1}{4}\; X\; 16 = 4mm$
ii>. $\ngtr 16mm = 16mm$

least value 8mm

II. Spacing/pitch
i>. $\ngtr 400mm$
ii>. $\ngtr 16\; x\; 16 = 256mm$ ~ 250 mm
iii>. $\ngtr 300mm$

least value 250 mm

Therefore provide the reinforcement of 8mm at 250mm c/c spacing

DESIGN OF SLABS

THEORY OF SLABS:

Slabs are plate elements forming floors and roofs of building and carrying distributed loads primarily by flexure. A slab may be supported by beams or walls and may be used as a flange of a T or L-beam. The common shapes of slabs are square, rectangle, triangular and circular.

Slabs are designed by using the theory of bending and shear. The following methodsof analysis are commonly used for the design of slabs.

- Elastic analysis-idealization in strips or beams
- Semi empirical coefficients as given in the code
- Yield line theory

➢ **Slabs are classified mainly into two types:**

1. One-way slabs
2. Two-way slabs

1. One-way slabs :

One-way slabs are those supported continuously on the two opposite sides so that the loads are carried along one direction only, in general when the aspect ratio $\frac{L_y}{L_x}$ is greater than 2. The direction in which the load is carried in one-way slabs is called the span. It may be in the long or short direction. One-way slabs are usually made to span in the shorter direction since the corresponding bending moments and shear forces are the least. The main Reinforcement is provided in the span direction. Steel is also provided in the transverse direction to distribute any unevenness that may occur in loading and to avoid the temperature and shrinkage effects in that direction. The steel is called distribution steel or secondary reinforcement. The main steel is calculated from the bending moment consideration and under no circumstances should it be less than the minimum specified by the code. The secondary reinforcement provided, is usually the minimum specified by the code for such reinforcement.

2. Two-way slabs :

Two-way slabs are those slabs that are supported continuously on all four sides and of such dimensions that the loads are carried to the supports along both directions. In two-way slabs, the slab is stiffened along both the directions by providing main steel reinforcement along both the directions. In general slabs are designed as two-way slabs when the aspect ratio L_x/L_y is less than 2. Generally two-way slabs are economical than one-way slabs.

Note:

1. When span is smaller than 4.5m in any direction, we can go for one way continuous slabs.
2. When span is larger than 4.5m in any direction, we can go for one way continuous slabs.

- The maximum permissible span length of slabs are considered as follows.

Support condition	**Cantilevers**		**Simply supported**		**Fixed / continuous**	
Slab type	One way	Two way	One way	Two way	One way	Two way
Max span(m)	1.5	2.0	3.5	4.5	4.5	6.0

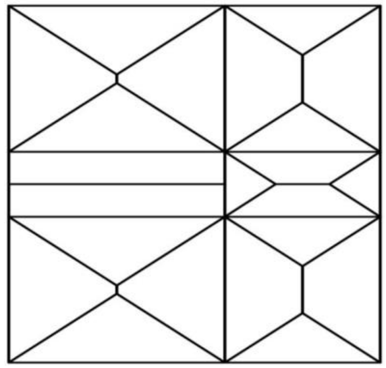
LOADING ON ONE-WAY SLAB:

When the slabs are supported on both edges, the total load will be distributed to both the edged proportions depending upon the support conditions. When the support conditions at both edges are either simply supported or continuous, then the load will be equally distributed to both the edges, and each equal to 0.5 W_u L is the total load on slab per meter width. When the support conditions at both the edges are different i.e., one is continuous and the other is simply supported then the load distributed to them will also be different, the load carried to the continuous edgewill be more because of the more stiffness due to more steel reinforcement than at simply supported edge. Here the load carried to the continuous edge is equal to 0.6 W_u L, whereas theload carried to the simply supported edge is equal to 0.4 W_u L.

IS-CODE PROVISIONS FOR DESIGN OF SLABS :

As per IS:456-2000, Code of practice for design of R.C.C structures recommends the following :

1. For frames the effective spans taken as per Clause No. 21.2of IS:456-2000
2. Effective depth is the distance between the centroid of the area of the tension reinforcement to the top of compression fiber excluding the finishing.
3. When L_y/L_x is greater than 2, the slab is designed as spanning one-way, when L_y/L_x is less than 2, the slab is designed as spanning two-way as per the coefficients given in table 22 of IS:456-2000 torsion reinforcement need not be provided at any corner contained by edges over both of which the slab is continuous.
4. Maximum diameter of reinforcing bar shall not exceed the 1/8th of the total thickness of slab (clause 25.2.2).
5. Cover to reinforcement, at each end of reinforcing bar not less than 25 mm nor less than twice the diameter of such bar (clause 25.4.1).
6. Cover to reinforcement, for tensile, compressive shear or other reinforcement in slab, not less than 20mm nor less than diameter of such slab.
7. Maximum permissible spacing of Main reinforcement shall not be more than 3 times of the effective depth of a slab or 30 cm, whichever is less.
8. Max, permissible spacing of distribution of reinforcement shall not be more than 5times the effective depth of a slab or 45 cm, whichever is less.
9. No shear reinforcement should be provided for slabs less than 200mm thick. However the increased value of shear resistance in slabs can be taken into account in design.
10. Minimum reinforcement in either direction in slab shall not be less than 0.15% of total cross-sectional area. However the value can be reduced to 0.12% when HYSD bars are used (clause 25.5.2.1).
11. Over the continuous edge of a middle strip the tension of the slab at a distance of 0.15L from the support and at least 50% extended to a distance of 0.3L.



Load Transfer from Slab to Beam

Reinforced concrete slab design and detailing guidelines for depth of slab, loads on slab, reinforcement guide for one-way and two-way slabs as per IS 456:2000 have been tried to present here.

Following are the RCC Slab Design and Detailing guidelines:

Reinforced Concrete Slab Design Guidelines

a) Effective span of slab:

Effective span of slab shall be lesser of the two

1. L = clear span + d (effective depth)
2. L = Center to center distance between the support

b) Depth of slab:

The depth of slab depends on bending moment and deflection criterion. the trail depth can be obtained using:

- Effective depth d= Span /((L/d)$_{Basic}$ x modification factor)
- For obtaining modification factor, the percentage of steel for slab can be assumed from 0.2 to 0.5%.
- The effective depth d of two way slabs can also be assumed using cl.24.1,IS 456 provided short span is <3.5m and loading class is <3.5KN/m^2

Type of support	Fe-250	Fe-415
Simply supported	L/35	L/28
Continuous support	L/40	L/32

Or, the following thumb rules can be used:

- One way slab d=(L/22) to (L/28).
- Two way simply supported slab d=(L/20) to (L/30)
- Two way restrained slab d=(L/30) to (L/32)

c) Load on slab:

The load on slab comprises of Dead load, floor finish and live load. The loads are calculated perunit area (load/m^2).

Dead load = D x 25 kN/m^2 (Where D is

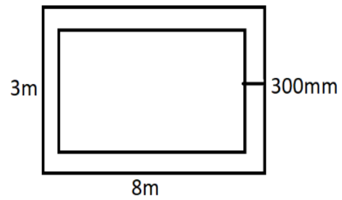
thickness of slab in m)Floor finish (Assumed

as)= 1 to 2 kN/m^2

Live load (Assumed as) = 3 to 5 kN/m^2 (depending on the occupancy of the building)

EXERCISE IV
ONE-WAY SLAB

Design a Simply Supported RCC slab with a clear dimensions of 3m×8m. Consider the width of the Supporting wall as 300mm, Weight of the weathering course is 1 Kn/m^2 (floor load), Live load as 2 Kn/m^2 by using M20 Grade of concrete & Fe415 steel. (Assume cover to be 25mm).



Given:

L_y = 8m, L_x =3m,

F_{ck}=20 N/mm^2, F_y=415 N/mm^2

$\frac{Ly}{Lx} = \frac{8}{3}$ =2.8m ≥2m (one way slab)

Step 1: Depth of the Slab

Effective depth ($d_{provided}$) $= \frac{L}{25} = \frac{3000}{25}$ = 120mm

D = d+ clear cover + ø/2 (or) d + effective cover

=120+25 = 145mm ≅ 150mm

Step 2: Effective Span Length

a. C-C distance = 3000+300 =3300mm
b. Clear span + effective depth =3000+120 =3120mm

Minimum of a, b = 3120mm.

Step 3: Calculate Design load (W_u), Shear Strength (V_u), Ultimate Moment (M_u)

Loading:

Dead Live (DL) = ɤ × b × D = 25×1×0.15 =3.75 Kn/m

Live load (LL) =2 kN/m

Floor Load (FL)=1 kN/m

Total Load = 3.75+2+1 =6.75 kN/m

Design Load /Factored Load (W_u) = 6.75 ×1.5 = 10.125 kN/m

Shear Strength (V_u) = $\frac{(Wu)\times Leffective}{2} = \frac{10.125\times 3.120}{2}$ = 15.795Kn/m

Ultimate Moment (M_u) = $\frac{(Wu)\times(L_{effective})^2}{8}$ = $\frac{10.125\times(3.12)^2}{8}$ =1232.01Kn/m^2

Step 4: Check for effective depth of slab

$M_{u\ limit}$ = 0.138×F_{ck}×b×d^2 ⇒ 1232.01= 0.138×20×1000×d^2

d $_{requried}$ =66.812 mm

Since d $_{requried}$ < d $_{provided}$ Hence OK

Step 5: Calculate Area of reinforcement (A_{st})

M_u =0.875$F_y$$A_{st}$d × [1-$\frac{f_y\times A_{st}}{f_{ck}\times b\times d}$]

A_{st} = 5483.221,299.9114349

= 299.911mm^2

$A_{st\ min}$ = 0.15×b×D = 0.15×1000×150 = 225mm^2

Since $A_{st\ min}$ < $A_{st\ calculated}$ Hence OK

Spacing of bars for main R/F

a) 3d = 3×125 =375

b) 300

c) $\frac{\frac{\pi}{4}\times d^2}{A_{st}}\times 1000$ = 261.744mm (Assume 10 mm dia of bar) $\cong 300mm$

Spacing = 300mm

Provide 10mm dia bars at 300mm c/c spacing

Spacing of bars for Distribution R/F

a) 5d = 5×125 =625

b) 450

c) $\frac{\frac{\pi}{4}\times d^2}{A_{st}}\times 1000$ = 160mm (Assume 8mm dia of bar) $\cong 160mm$

Spacing is min value =160mm ~ 150mm

Provide 10mm dia bars at 150 mm c/c spacing

Step 9: Check for Shear

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

=$\frac{15.795}{1000\times125}$ = 0.132 N/mm^2

% of reinforcement =$\frac{A_{st}}{bd}$×100 =$\frac{314}{1000\times125}$×100 = 0.26%

At support=0.26/2=0.13%

T_c =0.28 (Using IS456 2000 from Table 19 by knowing M20 grade & %of steel)

$T_{c\,max}$ =2.8 N/mm^2 (Using IS456 2000 from Table 20)

Since $T_v < T_c < T_{c\,max}$ Hence Ok

Step 10 : Check for deflection control

L/d $_{provided}$ = $\frac{3.125}{0.120}$ =26

L/d $_{Allowed}$ = (basic value ×MF)

f_{sc} =$0.58 f_y \times \frac{area\ of\ \frac{c}{s}\ of\ steel\ requried}{area\ of \frac{c}{s} steel\ provided}$ (from Pg 37)

$f_{sc} = 0.58 \times 415 \times \frac{299.9}{314} = 230 N/mm^2$

MF=1.5 (From IS 456 2000 fig 4)

L/d $_{Allowed}$ = 20×1.5 = 30

Since $L/d_{provided} < L/d_{Allowed}$ hence OK

EXERCISE-V

DESIGN OF TWO-WAY SLAB

Design a two-way slab with a clear dimensions of 3.5m×4m. Consider the width of the Supporting wall as 300mm, Weight of the weathering course is 1 Kn/m^2 (floor load), Live load as 3 Kn/m^2 by using M20 Grade of concrete & Fe415 steel. The edges of the slab are simply supported & corners are held down. (Assume effective cover to be 25mm).

Given:

L_y = 4m, L_x =3.5m,

F_{ck}=20 N/mm^2, F_y=415 N/mm^2

$\frac{Ly}{Lx} = \frac{4}{3.5}$ =1.14m <2m (one way slab)

Step 1: Depth of the Slab

Effective depth ($d_{provided}$)= $\frac{L}{28} = \frac{3500}{28}$ = 125mm

D = d+ clear cover + ø/2

= d+ effective cover

=125+25 = 145mm ≅ 150mm

Step 2: Effective Span Length

a. C-C distance = 3500+300 =3800mm
b. Clear span + effective depth =3500+125 =3625mm

Minimum of a, b =3625mm.

Step 3: Load calculation

Dead Live (DL) = γ × b × D

= 25×1×0.15

=3.75 Kn/m

Live load (LL) =3 Kn/m

Floor Load (FL)=1 Kn/m

Total Load = 3.75+2+1

=7.75 Kn/m

Step 4 : Design Moments

$M_x = \alpha_x \times wl_x^2$; $M_y = \alpha_y \times wl_x^2$

α_x :

$\frac{Ly}{Lx} = \frac{4}{3.5} = 1.14m$ [1.1→0.074 & 1.2→0.084]

$\alpha_x = 0.074 + \left[\frac{0.084-0.074}{1.2-1.1}\right] \times (1.14 - 1.1) = 0.078$

α_y :

$\frac{Ly}{Lx} = \frac{4}{3.5} = 1.14m$ [1.1→0.061 & 1.2→0.059]

$\alpha_y = 0.061 + \left[\frac{0.059-0.061}{1.2-1.1}\right] \times (1.14 - 1.1) = 0.062$

$M_x = \alpha_x \times wl_x^2$

$= 0.078 \times 11.625 \times 3.625 \times 3.625$

$= 11.91 KNm$

$M_y = \alpha_y \times wl_x^2$

$= 0.062 \times 11.625 \times 3.625^2$

$= 9.196 KNm$

Step 5: Check for effective depth of slab

$M_{u\ limit} = 0.138 \times F_{ck} \times b \times d^2 \Rightarrow 11.91 \times 10^6 = 0.138 \times 20 \times 1000 \times d^2$

$d_{requried} = 65.69$ mm

Since $d_{requried} < d_{provided}$ Hence OK

Step 6:

I. Calculate Main reinforcement (Along shorter span)

$M_{ux} = 0.875 F_y A_{st} d \times [1 - \frac{f_y \times A_{st}}{f_{ck} \times b \times d}]$

$A_{st} = 276.6 mm^2$

$A_{st\ min} = 0.12 \times b \times D = 0.12 \times 1000 \times 150 = 180 mm^2$

Since $A_{st\ min} < A_{st\ calculated}$ Hence OK

Spacing of bars for main R/F

a) $3d = 3 \times 125 = 375mm$

b) 300mm

c) $\frac{a_{st}}{A_{st}} \times 1000 = \frac{\frac{\pi}{4} \times d^2}{A_{st}} \times 1000$ = 181.7mm (Assume 8 mm dia of bar) $\cong 180mm$

Spacing is min value =180mm

II. Main reinforcement along longer span

Here d= D- d' -ø = 150-25-8 =117mm

M_{uy} $=0.875 F_y A_{st} d \times [1-\frac{f_y \times A_{st}}{f_{ck} \times b \times d}]$

$A_{st\ REQ} = 226.8mm^2$

Since $A_{st\ min} < A_{st\ calculated}$ Hence OK

Spacing of bars for main R/F

a) 3d = 3×125 =375mm

b) 300mm

c) $\frac{a_{st}}{A_{st}} \times 1000 = \frac{\frac{\pi}{4} \times d^2}{A_{st}} \times 1000$ = 221.6mm (Assume 8 mm dia of bar) $\cong 250mm$

Spacing is min value =250mm

Step 7: Reinforcement for Edge stirp [distribution steel]

$A_{st\ min} = 0.12 \times b \times D = 0.12 \times 1000 \times 150 = 180mm^2$

Spacing of bars for main R/F

a) 5d = 5×125 =625mm

b) 450mm

c) $\frac{a_{st}}{A_{st}} \times 1000 = \frac{\frac{\pi}{4} \times d^2}{A_{st}} \times 1000$ = 279.2mm (Assume 8 mm dia of bar) $\cong 270mm$

Spacing is min value =270mm

Step 8: Check for deflection

$L/d_{provided} = \frac{3.625}{0.125} = 29$

$L/d_{Allowed} = ((l/d)_{basic} \times Kt)$ [IS456 2000 Clause 23.2.1(a)]

$f_{sc} = 0.58 f_y \times \frac{area\ of\ \frac{c}{s}\ of\ steel\ requried}{area\ of \frac{c}{s} steel\ provided}$ (IS456 2000, Pg 37)

$f_{sc} = 0.58 \times 415 \times \frac{276.6}{279.2} = 230N/mm^2$ $\left[Ast_{provided} = \frac{1000}{spacing} \times ast = 279.2mm^2\right]$

% of reinforcement = $\frac{A_{st\ provided}}{bd} \times 100 = \frac{279.22}{1000 \times 125} \times 100 = 0.22\%$

Modification factor Kt=1.6

$L/d_{provided} = 20 \times 1.6 = 32$ (simply supported)

Since $L/d_{\ provided} < L/d_{\ Allowed}$ hence OK

EXERCISE-VI

DESIGN OF FOOTINGS

THEORY OF FOOTINGS:

Footing or Foundation is defined as the part of substructure, which transmits the load from superstructure to surrounding soil stratum safely.

- **Foundations are classified in to two types**.

1. **Shallow foundation**
2. **Deep foundation**

The depth of the foundation is less than or equal to the width of the foundation then thefoundation is said to be shallow foundation. If the depth of the foundation is greater than width of the foundation then the foundation is said to be Deep foundation. Design of footing mainly depends on the safe bearing capacity of the soil on which the footing rests and the load coming from the superstructure.

- **Footings may be Isolated, Combined :**

Isolated or independent footings are the footings that support the individual columns. They distribute and spread the load over a sufficiently large area of the soil stratum to minimize the bearing pressure. Isolated footings may be square, rectangular or circular.

In general, it is assumed that the soil behaves elastically i.e., the strain in the soil is proportional to applied stress i.e., stress and strain distribution in the soil immediately under the base of the footing is linear. Stress distribution is different for different soils.

For analysis purpose, a footing can be compared with a rigid body in equilibrium subjected to loads. Like other structural members, a footing is designed to resist shear forces and bending moments. In design, for any soil the pressure distribution is assumed to be uniform.

In design, the critical section for one way shear (beam shear) is at a distance equal to the effectivedepth, d from the face of column footing. The critical section for two way shear or slab type shearshall be at a distance d/2 from the periphery of column, perpendicular to the plane of the slab.

DESIGN OF COMBINED FOOTING

Design a combined footing for RC column A and column B, separated by a distance of 4m from centre to centre. Column A cross section is 500 mm^2 and carries a load of 1200kN. Column B cross section is 600mm^2 and carries a load of 1600kN. Soil bearing capacity is 200kN/m^2, fck is 20 N/mm^2, fy is 415 N/mm^2.

Solution

Step -1: find the dimensions

Area of footing = $\frac{Total\ load}{SBC}$

Total load:

i. Super imposed load = 1200 + 1600 = 2800kN
ii. Self-weight =10% of total load = 10% of 2800 = 280kN

Total load = 2800 + 280 = 3080kN

Area of footing = $\frac{3080}{200}$ = 15.4m^2

Dimensions – L * B

L > l (=4m)

Let the length of footing be 6m.

L * B = 15.4m^2

B = $\frac{15.4}{6}$ = 2.567 ≈ 2.6m

Let the centre of gravity (CG) of column load and CG of footing coincide at same point $\bar{x}$.

$\bar{x} = \frac{1600*4}{1200+1600}$ = 2.2857m

a_1 (dist. from edge to centre of column 1) $= \frac{L}{2} - \bar{x}$

$= \frac{6}{2} - 2.28 = 0.72$m

a_2 (dist. from edge to centre of column 2) = L – (l+ a_1)

= 6 – (4+0.72) = 1.28m

Dimensions of footing = L * B = 6*2.6 m^2

Step -2: SFD and BMD

Net upward pressure = $\frac{2800}{2.567*6}$

= 181.81 kN/m^2

Load per meter length width = 181.81 * 2.6 = 472.72kN/m

SFD left of column C1 = 472.72 * 0.72 = 340.35kN.

SFD right of column C1 = 1200 – 340.35 = 859.65kN.

SFD right of column C2 = 472.72 * 1.28 = 605.08kN.
SFD left of column C2 = 1600 – 605.08 = 994.91kN.

Taking BM at x $= \frac{859.65}{472.72}$ = 1.81m (472.72 * x = 859.65)
BM at C1 = 472.72*0.72*(0.72/2) =122.53kN-m.
BM at C2 = 472.72*1.28*(1.28/2) = 387.25kN-m.
(Taking BM at centre = 0 from left side of footing)

BM at centre of footing $= 1200 * 1.81 - \frac{472.72*(0.72+1.81)*(0.72+1.81)}{2}$
= 659.0769kN-m.
(Taking BM at face of C1 from left side of footing as 0)

BM at outer face of C1 $= \frac{472.72*(0.72-0.25)*(0.72-0.25)}{2}$
= 52.21kN-m.
(Taking BM at face of C2 from right side of footing as 0)

BM at outer face of C2 $= \frac{472.72*(1.28-0.3)*(1.28-0.3)}{2}$
= 227kN-m.

Depth, d = $\sqrt{\frac{Mu}{Ru*b}}$; Ru = 0.36*fck*($x_u$max/d) *(1-0.416*$x_u$max/d)
Ru = 0.36*20*0.48*(1-0.416*0.48)
= 2.761
d = $\sqrt{\frac{1.5*659.07*10^6}{2.761*2600}}$ = 371.1mm.

let d = 360mm
D = d+d’: d’ = 60(cover)
D = 360 + 60 = 420mm

Step -3: check for Punching shear

B_o
For C1 = b+d = 500+360 = 860mm
For C2 = 600+360 = 960mm

V = W – P_o*A = 1600 – 181.81*0.96^2
= 1432.44kN
Vu = 1.5*V = 1.5*1432.44 =2148.66kN.

$\tau v = \frac{Vu}{4*Bo*d} = \frac{2148.66*10^3}{4*960*360}$ = 1.55N/mm^2.

τc = 0.25*√fck = 0.25*√20 = 1.118n/mm^2.

$\tau v > \tau c$ “UNSAFE”

d = $\frac{Vu}{4*Bo*\tau c} = \frac{2148.66*10^3}{4*960*1.118}$ = 500.48mm.

Let d = 500mm
D = 500 + 60 = 560mm

Step -4: Design of bending tension

1. Max. Hogging Moment

Mu = 1.5*Mu at max =1.5*659.08 = 988.62kN-m.

Ast = $\frac{0.5*fck}{fy}(1-\sqrt{1-\frac{4.6*Mu}{fckbd^2}}$)*bd

= $\frac{0.5*20}{415}(1-\sqrt{1-\frac{4.6*988.62*10^6}{20*2600*500^2}}$)*2600*500 = 6066.495mm^2.

Using 16mm dia bars as reinforcement

No of bars = $\frac{6066.495}{\frac{\pi}{4}*16^2}$ =30.17 ≈31.

% of steel

Pt = $\frac{Ast}{b.d}*100 = \frac{31*\frac{\pi}{4}*16^2}{2600*500}*100$

= 0.479%.

Vu = 1.5*V at POC
= 1.5*749.09
= 1129.53kN.

Xu = $\frac{0.87*fy*Ast}{0.36*fck*B}$
=120.15mm

Mu =0.87*fy*Ast*(d-0.416*Xu) = 1012.65kN-m

Check, from IS 456:2009, pg. 44

$$\frac{Mu}{Vu}+Lo>Ld$$

Lo = 12* dia of bars = 12*16 = 192 or 500 (max of both)

Ld = 47* dia of bars = 47*16 = 752mm

$$\frac{1012.65}{1129.53}+500>752$$

1402.23 > 752, "OK"

2. Reinforcement at sagging moment

a. Mu = 1.5*227 = 340.5kN-m. (at C2)

Ast $= \frac{0.5*fck}{fy}(1 - \sqrt{1 - \frac{4.6*Mu}{fckbd^2}})$*bd

= 1947.66mm^2.

Using 12mm dia bars as reinforcement

No of bars = $\frac{1947.66}{\frac{\pi}{4}*12^2}$ =17.19 ≈18

% Of steel

Pt = $\frac{Ast}{b.d} * 100$ =$\frac{18*\frac{\pi}{4}*12^2}{2600*500} * 100$

= 0.156%.

Vu = 1.5*V at POC = 1.5*749.09

= 1129.53kN.

Xu = $\frac{0.87*fy*Ast}{0.36*fck*B}$ =39.29mm

Mu =0.87*fy*Ast*(d-0.416*Xu) = 355.5kN-m

Check, from IS 456:2009, pg. 44

$$\frac{Mu}{Vu} + Lo > Ld$$

Lo = 12* dia of bars = 12*12 = 144 or 500 (max of both)

Ld = 47* dia of bars = 47*12 = 752mm

$$\frac{355.5}{1129.53} + 500 > 752$$

816.38 > 752, "OK"

b Mu = 1.5 * 52.22 = 78.31kN-m. (at C1)

Ast = $\frac{0.5*fck}{fy}(1 - \sqrt{1 - \frac{4.6*Mu}{fckbd^2}})$*bd = 437.08mm^2.

Ast, min = 0.12%bD = $\frac{0.12}{100} * 2600 * 560$

=1747mm^2.

Ast, min > Ast, provide Ast, min steel.

Using 12mm dia bars as reinforcement

No of bars = $\frac{1747}{\frac{\pi}{4}*12^2}$ =15.44 ≈16

Step -5: Transverse reinforcement

1. C1

Projection of a = 0.5*(B-b) = 0.5*(2600-500)

= 1050mm.

B_1 = b+2*d = 500+ 2*500 = 1500mm.

Net upward pressure, Po= $\frac{W}{B*B1}$ = = $\frac{1200}{2600*1500}$ = 307.7kN/m^2.

M = 307.7 *1*$\frac{a^2}{2}$ = 307.7*1*$\frac{1.05^2}{2}$ = 169.62kN-m.
Mu = 1.5 * M = 254.43kN-m.
d = $\sqrt{\frac{254.43*10^6}{2.761*2600}}$ = 303.56mm

we take d as
500 – 0.5 *dia of bars at C1 – 0.5*dia of bars at C2(sagging moments)
=500-6-6 = 488mm
Ast = $\frac{0.5*fck}{fy}(1-\sqrt{1-\frac{4.6*Mu}{fckbd^2}})$*bd = 1546mm2 (take d = 488mm)

Using 12mm dia bars
Spacing = $\frac{1000*ast}{Ast}$ =73.15mm

Let us provide 70mm centre to centre spacing.

2. C2
Projection of a = 0.5*(B-b) = 0.5*(2600-600) = 1000mm.
B_1 = b+2*d = 600+ 2*500 = 1600mm.

Net upward pressure, Po= $\frac{W}{B*B1}$ = = $\frac{1600}{2600*1600}$ = 784.61kN/m^2.
M = 784.61*1*$\frac{a^2}{2}$ = 784.61*1*$\frac{1^2}{2}$ = 192.30kN-m.
Mu = 1.5 * M = 254.43kN-m.

Ast = $\frac{0.5*fck}{fy}(1-\sqrt{1-\frac{4.6*Mu}{fckbd^2}})$*bd = 1814.92mm2 (take d = 488mm)

Using 12mm dia bars
Spacing = $\frac{1000*ast}{Ast}$ =62.31mm

Let us provide 60mm centre to centre spacing.

Step -6: check for one way shear and diagonal shear

1. C2
Let us assume that critical section be at d distance from face of column.
Distance from centre of column = 600/2 + 500 =800mm.
L	At the point of contra flexure is at a point x from point d, centre of column
A

Point of contraflexure:
1200*x-466.67*(0.72+x) *(0.72+x)/2 = 0
X= 0.1486 mm

Vu = 994.41 – 0.8*472.72 = 925.1kN

$\tau v = \frac{Vu}{B.d} = \frac{925.1*10^3}{2600*500}$ = 0.712%.

From IS 456:2000, clause 40.2.1.1, pg. 72, since the depth of column > 300mm

k = 1; k*τc = 1*0.47 = 0.47N/mm^2. (τc from table 19, pg. 73)

τv > τc "UNSAFE", "design shear reinforcement."

Using 8-legged stirrup bars of 8mm as dia.

Sv = $\frac{0.87*fy*Asv*d}{Vus}$;

Asv = single bar area = 8*($\frac{\pi}{4}$*8^2) = 402.123mm^2.

Sv = 80mm

Provide spacing of 80mm from centre to centre.

2. Diagonal shear (at POC) C2

Vu = 1.5*(994.91-0.52*472.72)

= 1123.64kN

$\tau v = \frac{Vu}{B.d} = \frac{1123.64*10^3}{2600*500}$ = 0.86%.

From IS 456:2000, clause 40.2.1.1, pg. 72, since the depth of column > 300mm

k = 1; k*τc = 1*0.284 = 0.284N/mm^2. (τc from table 19, pg. 73)

τv > τc "UNSAFE"

Sv = $\frac{0.87*fy*Asv*d}{Vus}$; Asv = single bar area

= 64.23mm

Provide spacing of 60mm from centre to centre.

3. C1

Vu = 1.5*(859.63-0.14*472.72) = 1149.90kN.

From IS 456:2000, clause 40.2.1.1, pg. 72, since the depth of column > 300mm

k = 1; k*τc = 1*0.284 = 0.284N/mm^2. (τc from table 19, pg. 73)

τv > τc "UNSAFE"

Sv = $\frac{0.87*fy*Asv*d}{Vus}$; Asv = single bar area

= 63.09mm

Provide spacing of 60mm from centre to centre.

EXERCISE-VII

ANALYSIS OF SLAB BRIDGE

A Bridge is a structure imparting passage over partner obstacle at the same time as not remaining the method at a lower vicinity. The required passage can also be for a street, a railway, pedestrians, a canal or a pipeline. The obstacle to be crossed can be a river, a street, railway or a valley.

Bridges variety in period from a few meters to several kilometers. They are among the largest systems built with the aid of man. The demands on design and on substances are very excessive. A bridge should be robust enough to support its personal weight in addition due to the fact the burden of the individuals and cars that use it.

Numbers of bridges have a concrete, metallic, or wood framework & an asphalt or concrete route on which individuals and vehicles travel. The T-beam Bridge is a long way and away the most unremarkably followed type in the span range of ten to 20-five meter.

The shape is so named because of the foremost longitudinal girders analyses & designed as T-beams imperative with a region of the deck block, that's cast monolithically with the girders. Simply supported T-beam span of over thirty meters are rare due to the fact the loading then turns into too critical.

Components of a Bridge

The Superstructure consists of the following components:

1. Deck slab
2. Cantilever slab element
3. Footpaths, if provided, kerb and handrails or crash limitations.
4. Longitudinal girders taken into consideration in the layout to be of T-section
5. Cross beams or diaphragms, intermediate and give up ones.
6. Wearing coat
7. Cross beams or diaphragms, intermediate and cease ones
8. Wearing coat

The Substructure consists of the following structures:

1. Abutments at the intense ends of the bridge.
2. Piers at intermediate helps in case of a couple of span bridges.
3. Bearings and pedestals for the decking.
4. Foundations for each abutments

RC SLAB CULVERT

Design a reinforced concrete slab culvert for National highway to suit the following data:

a. Carriageway – two lane (7.5 m wide)

b. Foot paths – 1 m on either side

c. Clear span – 6 m

d. Wearing coat = 80 mm

e. Width of bearing = 0.4 m

f. Materials – M25 grade concrete and Fe415 grade HYSD bars

g. Loading – IRC Class AA tracked vehicle

Design the RC deck slab and sketch the details of reinforcement in the longitudinal and cross section of slab.

1. Data

a. Clear span = 6 m

b. Width of bearing = 0.4 m

c. IRC Class AA tracked vehicle loading

d. M25 grade concrete and Fe415 grade HYSD bars

2. Permissible Stresses

$\sigma_{cb} = 8.3\ N/mm^2$	$m = 10$	$Q = 1.1$
$\sigma_{st} = 200\ N/mm^2$	$j = 0.9$	

3. Depth of slab and effective span

Assume thickness of slab at 80 mm per meter of span for highway bridge

Therefore, overall slab thickness = 80 x 6 = 480 mm

Adopt overall depth of 500 mm for the slab

Using 20 mm diameter HYSD bars with clear cover of 30 mm.

Effective depth = 500 – (30 + 10) = 460 mm

Width of bearing = 400 mm

Effective span is the least of

a) (clear span + effective depth) = (6 + 0.46) = 6.46 m

b) Centre to centre of bearing = (6 + 0.4) = 6.4 m

Effective span = L = 6.4 m

The cross section of the deck slab is shown in Fig.

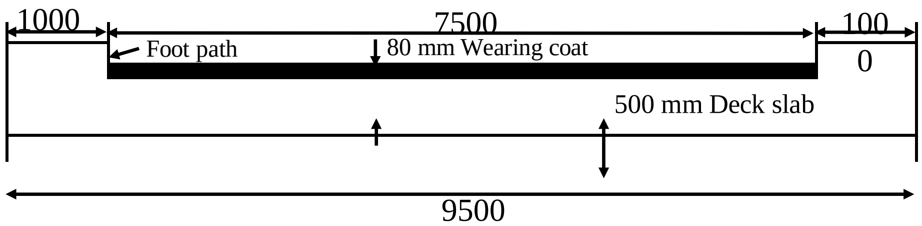


Fig. Cross – section of the Deck Slab

4. Dead Load Bending Moments

Dead weight of slab = (0.5 x 24) = 12 kN/m^2

Dead weight of wearing coat = (0.08 x 22) = 1.76 kN/m^2

Therefore, Total load = 14 kN/m^2

Dead load bending moment = $\dfrac{14x6.4^2}{8}$

= 72 kN.m

5. Live Load Bending Moment

Generally, the bending moment due to live load will be maximum for IRC class AA tracked vehicle. Impact factor for class AA tracked vehicle is 25% for 5 m span, decreasing linearly to 10% for 9 m span.

Therefore for 6.4 m span, Impact factor = 19.7%

The tracked vehicle is placed symmetrically on the span.

Effective length of load = Length of the tyre contact + (2 x overall thickness of the deck including the thickness of the wearing coat)

Effective length of load = 3.6 + 2 x (0.5 + 0.08) = 4.76 m

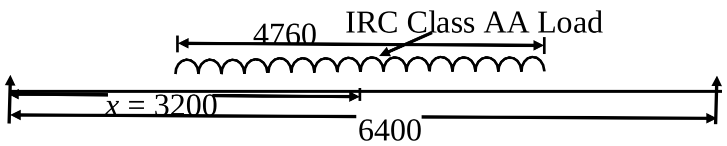


Fig. Position of Load for Maximum Bending Moment

Effective width of slab, perpendicular to span is expressed as,

$$b_e = K\ x\ [(1 - \frac{x}{L})] + b_w$$

Where,

b_e = Effective width od slab over which load is effective

L = Effective span of the simply supported slab

x = distance of centre of gravity of the concentrated load form the nearest support

K = a constant having values depending on B/L values (Appended in Table 7.1)

b_w = Width of the dispersion area of the wheel load on the slab through the wearing coat.

This is given by (w + 2h), where h is the thickness of the wearing coat, w is the contact width of the wheel on the slab perpendicular to the direction of movement.

$x = 3.2\ m, L = 6.4\ m, B = 9.5\ m, (\frac{B}{L}) = 1.48$

$b_w = (0.85 + 2\ x\ 0.08) = 1.01\ m$

From Table 7.1, For (B/L) = 1.48

Simply supported slab, K = 2.84

$b_e = 2.84\ x\ 3.2\ (1 - \frac{3.2}{6.4}) + 1.01 = 5.56\ m$

The tracked vehicle is placed close to the Kerb with the required minimum clearance as shown in Fig.

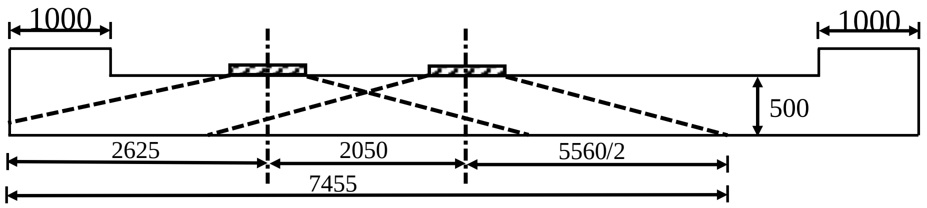


Fig. Effective width of dispersion of IRC Class AA loads

Net effective width of dispersion = 7.455 m

Total load of two tracks with impact = 700 x 1.197 = 838 kN

Average intensity of load = $\frac{838}{4.76\ x\ 7.455}$

= 23.61 kN/m^2

Maximum bending moment due to live load is given by,

$$M_{max} = [\frac{23.61\ x\ 4.76}{2}\ x\ 3.2] - [\frac{23.61\ x\ 4.76}{2}\ x\ \frac{4.76}{4}] = 113\ kN.m$$

Therefore, total design *B. M.* = (113 + 72) = 185 kN.m

6. Shear due to class AA Tracked Vehicle

For maximum shear at support, the IRC class AA tracked vehicle is arranged as shown in Fig.

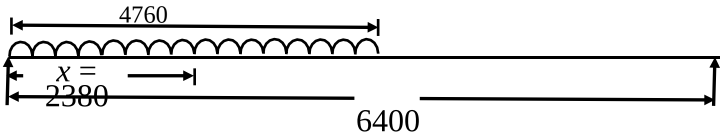


Fig. Position of Load for Maximum Shear

Effective width of dispersion is given by

$$b_e = K\ x\ [(1 - \frac{x}{L})] + b_w$$

Where x = 2.38 m

B = 9.5 m $\quad$ (B/L) = 1.48

L = 6.4 m $\quad$ K = 2.84

$b_w = 1.01\ m$

Therefore,

$$b_e = 2.84\ x\ 2.38\ \left(1 - \frac{2.38}{6.4}\right) + 1.01 = 5.256\ m$$

Therefore, width of dispersion = (2625 + 2050 + 5256/2) = 7303 mm

$$\text{Average intensity of load} = \frac{838}{4.76\ x\ 7.303}$$

$$= 24.1\ \text{kN/m}^2$$

$$\text{Therefore, shear force } VA = \frac{24.1\ x\ 4.76\ x\ (6.4 - 2.38)}{6.4} = 72\ kN$$

$$\text{Dead Load Shear} = \frac{14\ x\ 6.4}{2} = 45\ kN$$

Therefore, Total design shear = (72 + 45) = 117 kN

7. Design of Deck Slab

Effective depth required is

$$d = \sqrt{\frac{M}{Q.b}} = \sqrt{\frac{185\ x\ 10^6}{1.1\ x\ 1000}} = 410\ mm$$

Effective depth provided = 460 mm

Therefore, $A_{st} = \dfrac{M}{\sigma_{st}\ x\ j\ x\ d} = \dfrac{185\ x\ 10^6}{200\ x\ 0.9\ x\ 460} = 2234\ mm^2$

Spacing of 20 mm diameter bars $= \dfrac{1000\ x\ 314}{2234} = 140mm$

Adopt 20 mm diameter HYSD bars at 140 mm centers as main reinforcement to conform to the criteria of control of cracking according to IRC:21 – 1987.

Bending moment for distribution steel is

$= (0.3M_L + 0.2M_D) = 0.3\ x\ 113 + 0.2\ x\ 72 = 49\ kN$

Using 12 mm diameter bars,

Effective depth = 460 – (10 + 6) = 444 mm

Therefore, $A_{st} = \dfrac{M}{\sigma_{st}\ x\ j\ x\ d} = \dfrac{49\ x\ 10^6}{200\ x\ 0.9\ x\ 444} = 613\ mm^2$

Spacing of 20 mm diameter bars $= \dfrac{1000\ x\ 314}{613} = 184mm$

Provide 12 mm diameter HYSD bars at 150 mm centres as distribution steel

Check for Shear Stresses

As per IRC:21 – 1987, shear stresses in the slab are checked as follows:

Design Shear stress $r = \dfrac{V}{b.d}$

where, V = Design shear force

b = width of section

d = effective depth

$$r = \frac{V}{b.d} = \frac{117\ x\ 10^3}{1000\ x\ 460} = 0.254\ N/mm^2$$

Permissible shear stress in slabs without shear reinforcement is computed as,

$$r_c = k_1. k_2. r_{co}$$

where, r_c= The permissible shear stress

k_1= (1.14 – 0.7 d) ≥ 0.5 where *'d'* is expressed in meters

k_2 = (0.5 + 0.25 ρ) ≥ 1

r_{co}= Basic value given below different grade of concrete in Table

ρ = Percentage of longitudinal reinforcement ratio = (100 *As*/bd)

As = Area of longitudinal reinforcement which continues at least *'d'* beyond the section considered or fully anchored when support section is considered.

b = width of the section

Table Basic Values of Permissible Shear Stress (r_{co})

Grade of Concrete	M – 15	M – 20	M – 25	M – 30	M – 35	M – 40
r_{co}	0.28	0.34	0.4	0.45	0.5	0.5

Assuming 20 mm diameter bars spaced at 280 mm at support section (alternate bars bent up)

$$A_s = \frac{1000\ x\ 314}{280} = 1121\ mm^2$$

$$\rho = \frac{100\ x\ 1121}{1000\ x\ 460} = 0.243$$

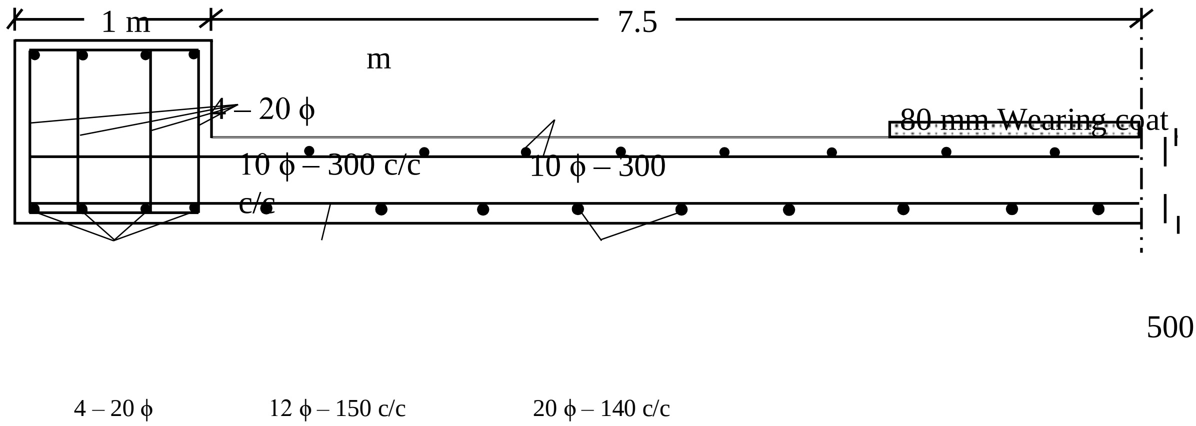
$$k_2 = (0.5 + 0.25\ x\ 0.243) = 0.560 \geq 1$$

$$k_1 = (1.14 - 0.7\ x\ 0.460) = 0.82 \geq 0.5$$

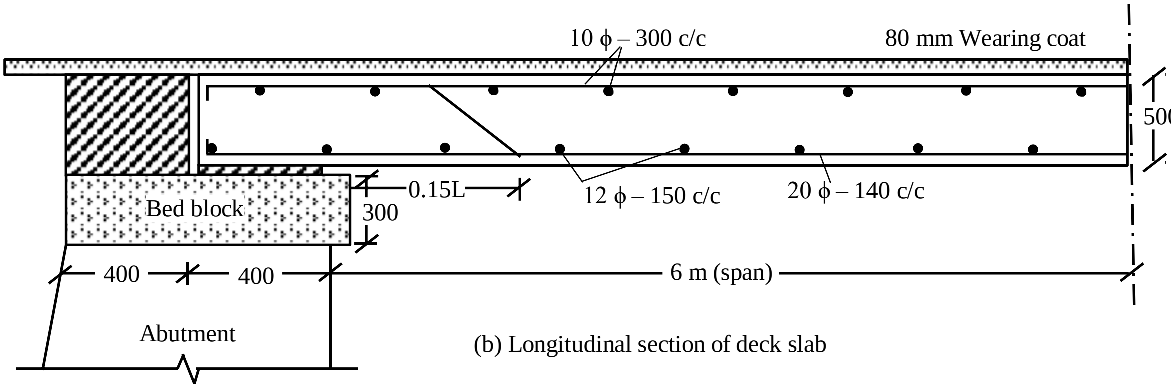
For M25 grade of concrete, $r_{co} = 0.40\ N/mm^2$

$$r_c = k_1 . k_2 . r_{co} = 0.82\ x\ 1\ x\ 0.40 = 0.328\ N/mm^2$$

Since $r < r_c$, the shear stresses are within safe permissible limits.



(a) Cross section of deck slab



(b) Longitudinal section of deck slab

EXERCISE-VIII
DESIGN AND DETAILING OF RETAINING WALLS

Retaining walls are usually built to hold back soil mass. However, retaining walls can also be constructed for aesthetic landscaping purposes
Classification of Retaining walls
• Gravity wall-Masonry or Plain concrete
• Cantilever retaining wall-RCC (Inverted T and L)
• Counterfort retaining wall-RCC
• Buttress wall-RCC

Earth Pressure:
Earth pressure is the pressure exerted by the retaining material on the retaining wall. This pressure tends to deflect the wall outward.
Types of earth pressure :

1. Active earth pressure or earth pressure (Pa)
2. Passive earth pressure (Pp).

Active earth pressure tends to deflect the wall away from the backfill.

Analysis for dry back fills:

- Maximum pressure at any height, p=k a h
- Total pressure at any height from top, P_a=1/2[k_a Γ h]*h = [ka Γ h2]2

- Bending moment at any height
  - M= P_a x h/3= [k_a Γ h^3] / 6

- Total pressure, P_a = [k_a Γ h^2] / 2

- Total Bending moment at bottom,
  - M = [k_a Γ h^3] / 6

Where, k_a = Coefficient of active earth pressure
= (1-sinΦ)/(1+sinΦ)= $\tan^2 \Phi$
= 1/k_p, coefficient of passive earth pressure
Φ = Angle of internal friction or angle of repose
Γ =Unit weight or density of backfill

Note: If = 30, k_a=1/3 and k_p =3. Thus k_a is 9 times k_p

Stability requirements of RW :
• It should not overturn
• It should not slide
• It should not subside, i.e Max. pressure at the toe should not exceed the safe bearing capacity of the soil under working condition

1. **Check against overturning:**

Factor of safety against overturning

$= M_R / M_O$ 1.55 (=1.4/0.9)

Where,

M_R = Stabilizing moment or restoring moment

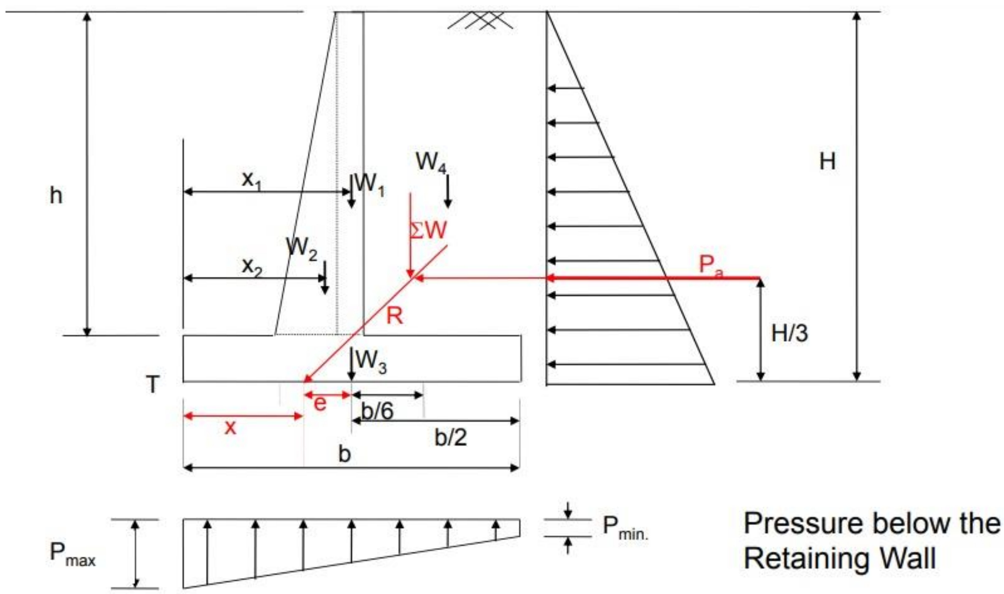
M_O =overturning moment

2. **Check against Sliding**:

FOS against sliding = [Resisting force to sliding] / [Horizontal force causing sliding]

= W/Pa 1.55 (=1.4/0.9)

Note: In case the wall is unsafe against sliding, provide “Shear Key”



Let the resultant R due to W and P_a lie at a distance x from the toe.

X = M / W,

M = sum of all moments about toe.

Eccentricity of the load = e = (b/2 - x) < b/6

Minimum pressure at heel = P_{min} = Wb1 - 6eb > zero

For zero pressure, e=b/6, the resultant should cut the base within the middle third.

Maximum pressure at toe = P_{max} = Wb1 + 6eb < SBC

Depth of Foundation

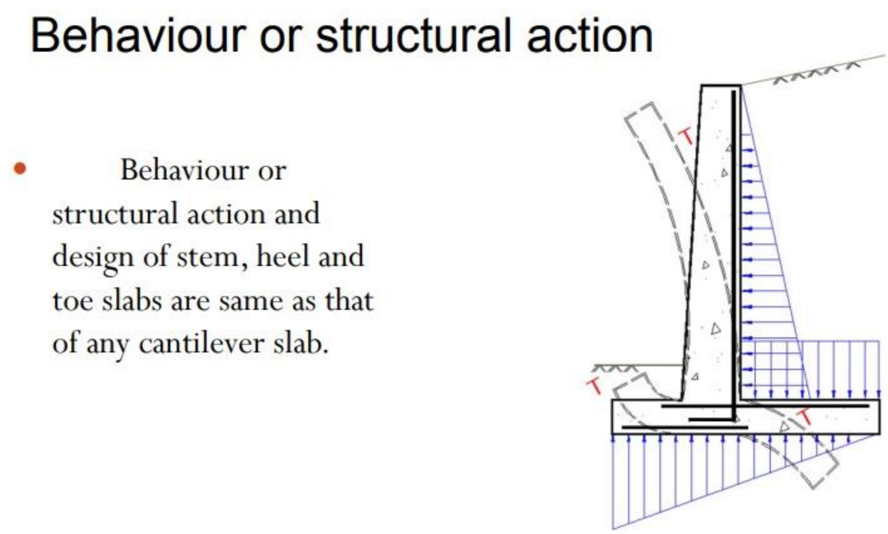
Rankine’s formula:

$$Df = SBC\Gamma 1-\sin\Phi 1+\sin\Phi^2$$

$$= SBC\Gamma K_a^2$$

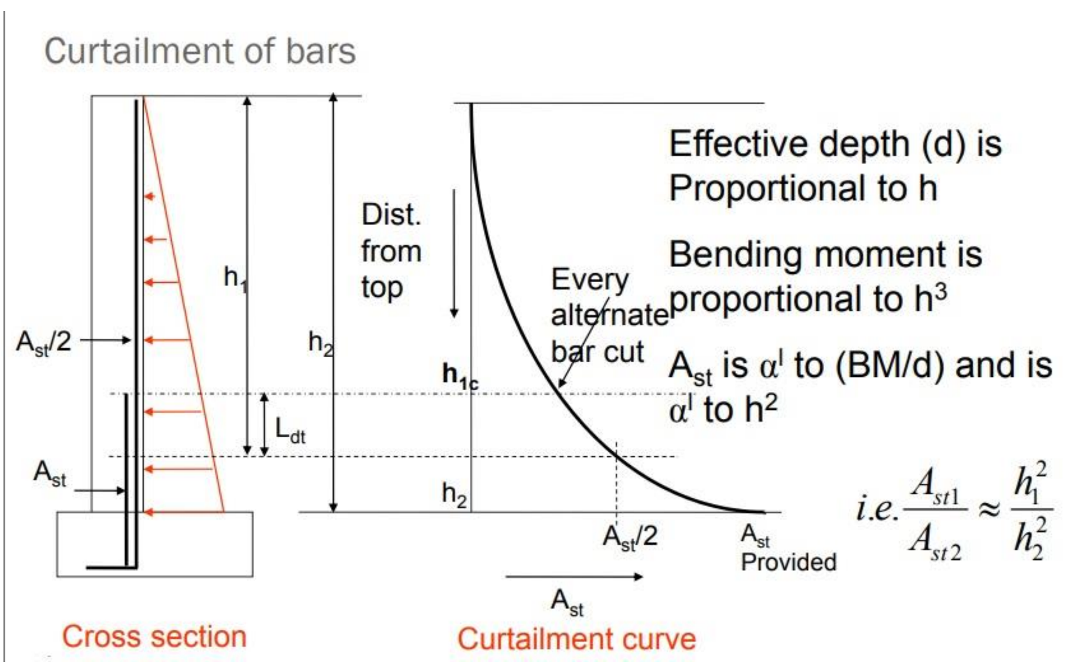
Preliminary Proportioning (T shaped wall)

- Stem: Top width 200 mm to 400 mm
- Base slab width b= 0.4H to 0.6H, 0.6H to 0.75H for surcharged wall
- Base slab thickness= H/10 to H/14
- Toe projection= (1/3-1/4) Base width



Design of Cantilever RW:

- Stem, toe and heel acts as cantilever slabs
- Stem design: $M_u = psf\ [k_a \Gamma h^3] / 6$
- Determine the depth d from $M_u = M_{u,lim} = Qbd^2$
- Design as balanced section or URS and find steel
- $M_u = 0.87 f_y A_{st}[d - f_y A_{st}/(f_{ck} b)]$



Design of Heel and Toe

1. Heel slab and toe slab should also be designed as cantilever. For this stability analysis should be performed as explained and determine the maximum bending moments at the junction.
2. Determine the reinforcement.
3. Also check for shear at the junction.
4. Provide enough development length.
5. Provide the distribution steel

Design a cantilever retaining wall (T type) to retain earth for a height of 4m. The backfill is horizontal. The density of soil is 18kN/m3. Safe bearing capacity of soil is 200 kN/m2. Take the coefficient of friction between concrete and soil as 0.6. The angle of repose is 30°. Use M20 concrete and Fe415 steel.

Data:

h' = 4m, SBC= 200 kN/m^2, Γ = 18 kN/m^3, μ=0.6, φ=30°

Depth of foundation

To fix the height of retaining wall [H]

H= h' +D_f

Depth of foundation

Df = SBCΓ1-sinΦ1+sinΦ^2

= 1.23m say 1.2m

Therefore, H= 5.2m

Proportioning of wall

Thickness of base slab= (1/10 to 1/14)H

= 0.52m to 0.43m, *say 450 mm*

Width of base slab =b = (0.5 to 0.6) H

=2.6m to 3.12m, *say 3m*

Toe projection= pj = (1/3 to ¼)H

=1m to 0.75m say 0.75m

Provide 450 mm thickness for the stem at the base and 200 mm at the top

1. **Design of Stem**

P_h = ½ x 1/3 x 18 x 4.75^2

=67.68 kN

M = P_h h / 3 = 0.333 x 18 x 4.75^3/6

= 107.1 kN-m

M_u = 1.5 x M

= 160.6 kN-m

Taking 1m length of wall,

M_u/bd^2= 1.004 < 2.76, URS (Here d=450- eff. Cover=450-50=400 mm)

To find steel P_t=0.295% <0.96%

A_{st}= 0.295 x 1000 x 400/100 = 1180 mm^2

#12 @ 90 < 300 mm and 3d, *OK*
A_{st} provided= 1266 mm^2 [0.32%]

Curtailment of bars in stem

- Curtail 50% steel from top
- $(h_1/h_2)^2$ = 50% / 100% = ½
- $(h_1/4.75)^2$ = ½, h_1 = 3.36m

Actual point of cutoff = 3.36-Ld=3.36-47 φbar
= 3.36- 0.564
= 2.74m from top.
Spacing of bars = 180 mm c/c < 300 mm and 3d *OK*

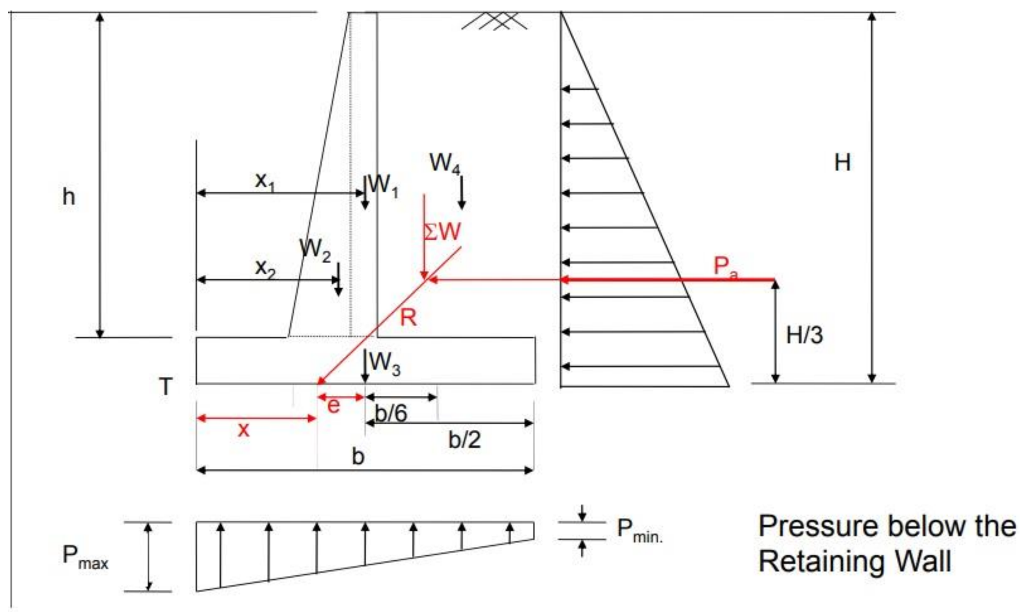
Development length (Stem steel):
L_d=47 φbar =47 x 12 = 564 mm
Secondary steel for stem at front :
0.12% A_G = 0.12x450 x 1000/100 = 540 mm2 #10 @ 140 < 450 mm and 5d *OK*

Distribution steel = 0.12% A_G = 0.12x450 x 1000/100
= 540 mm2
#10 @ 140 < 450 mm and 5d ok

Check for shear
Max. SF at Junction of stem and base
= Ph=67.68 kN
Ultimate SF = Vu=1.5 x 67.68 = 101.52 kN
Nominal shear stress =ζv = Vu/bd
= 101.52 x 1000 / 1000x400
= 0.25 MPa
To find ζc: $100A_{st}/bd$ = 0.32%,
From IS:456-2000, ζc= 0.38 MPa ζv < ζc, Hence safe in shear.

Stability Analysis:

Load	**Magnitude, kN**	**Distance from A, m**	**BM about A kN-m**
Stem W1	0.2x4.75x1x25 = 23.75	1.1	26.13
Stem W2	½ x0.25x4.75x1x25 = 14.84	0.75 + 2/3x0.25 =0.316	13.60
B. slab W3	3.0x0.45x1x25=33.75	1.5	50.63
Back fill, W4	1.8x4.75x1x18 = 153.9	2.1	323.20
Total	**ΣW= 226.24**		**ΣMR=413.55**
Earth Pre. =PH	PH =0.333x18x5.22/2	H/3 =5.2/3	**MO=140.05**



Stability checks

Check for overturning

FOS = $\Sigma M_R / M_O$= 2.94 >1.55 safe

Check for Sliding

FOS = $\mu\ \Sigma W / P_H$= 2.94 >1.55 safe

Check for subsidence

X=$\Sigma M / \Sigma W$= 1.20 m > b/3

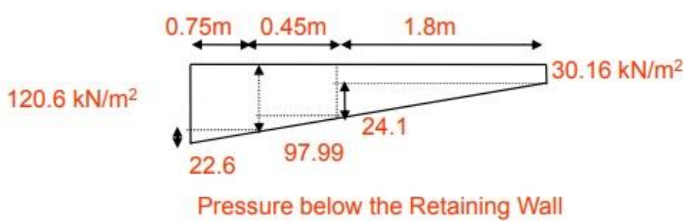
e= b/2 –x = 3/2 – 1.2 = 0.3m < b/6

Pressure below the base slab

P_{Max}=120.66 kN/m^2 < SBC, safe

P_{Min} = 30.16 kN/m^2 > zero, No tension or separation, safe

2. Design of Heel:



Load	Magnitude, kN	Distance from C, m	BM_C kN-m
Backfill	153.9	0.9	138.51
Heel Slab	0.45x1.8x25 = 27.25	0.9	18.23
Pressure dist. rectangle	30.16 x 1.8 =54.29	0.9	-48.86
Pressure dist. triangle	½ x 24.1 x1.8=21.69	⅓ x 1.8	-13.01
Total	**ΣW= 226.24**		**ΣMR= 94.86**

- M_u = 1.5 x 94.86 =142.3 kNm
- M_u / bd^2 = 0.89 < 2.76, URS
- Pt =0.264% < 0.96%
- A_{st} = 0.264x1000x400/100
  =1056 mm 2

  #16@ 190 < 300 mm and 3d ok
  Ast provided= 1058mm [0.27%]

OR M_u =0.87 fy Ast[d - (fy Ast/fckb)]

Development length: Ld=47 φbar =47 x 16 = 752mm
Distribution steel: #10 @ 140 < 450 mm and 5d *OK*

Check for shear at junction (Tension) :
Maximum shear = V =105.17 kN,
$V_{U,max}$ = 157.76 kN,

Nominal shear stress = ζ v =Vu/bd
= 101.52 x 1000 / 1000x400
= 0.39 MPa

To find ζ c: 100Ast/bd = 0.27%,
From IS:456-2000, ζ c= 0.37 MPa
ζ v slightly greater than ζc, *Hence slightly unsafe in shear*

3. Design of Toe

Load	Magnitude, kN	Distance from C, m	BM_C kN-m
Toe Slab	0.75x0.45x25	0.75/2	-3.164
Pressure dist. rectangle	97.99x0.75	0.75/2	27.60
Pressure dist. triangle	½ x22.6 x1.0.75	2/3x1=0.75	4.24
Total			**ΣM=28.67**

- M_u = 1.5 x 28.67 =43 kN-m
- M_u / bd^2 = 0.27< 2.76, URS
- Pt is very small provide minimum A_{st} (0.12% $A_{Gt\backslash}$)
- A_{st} =540 mm^2

#10 @ 140 < 300 mm and 3d ok

Development length: L d=47 φbar =47 x 10 = 470 mm

Check for shear at junction (Tension) :

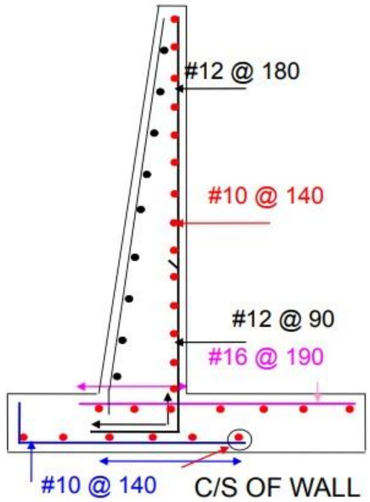
Net shear = V =(120.6+110.04)/2 x 0.35 - 0.45x0.35x25
=75.45kN

$V_{U,max}$ = 75.45x1.5=113.18 kN

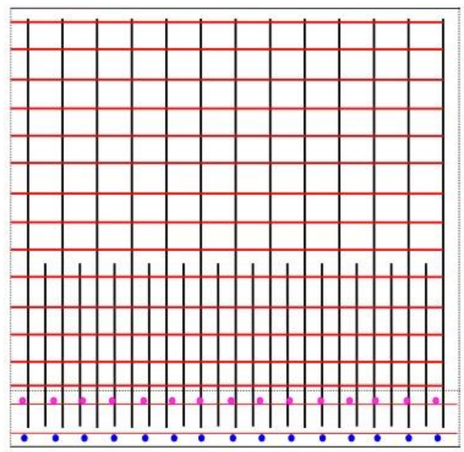
Nominal shear stress = ζ v =Vu/bd
= 113.17x1000/(1000x400)
=0.28 MPa

pt ≤0.25%, From IS:456-2000, ζ c= 0.37 MPa
ζ v < ζc, Hence safe in shear.

Drawing and detailing



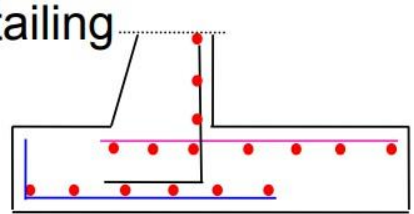
C/S OF WALL



L/S ELEVATION OF WALL

Drawing and detailing

BASE SLAB DETAILS



PLAN OF BASE SLAB