DESIGN OF SLAB

Data:

Concrete	M20	fck	=	20	N/mm^2
Steel	Fe500	fy	=	500	N/mm^2
Shorter Span		Lx	=	2.97	m
Longer Span		Ly	=	5.26	m
Assuming Depth of Slab		D	=	150	mm
Assuming Dia of reinf			=	8	mm
Clear Cover			=	20	mm
Eff.Depth of Slab		d	=	126	mm
Eff.Shorter Span		L_x	=	2.97	m
Eff.Longer Span		L_y	=	5.26	m

1.Load Calculations :

Self weight of slab	(0.15 x 20.00)	=	3.00	kN/m^2
Ceiling Plastering		=	0.25	kN/m^2
Floor Finish		=	1.00	kN/m^2
Weathering Course (Weight Not Applicable)		=	0.00	kN/m^2
Live load		=	2.00	kN/m^2
Total Load		=	7.00	kN/m^2
Load Factor		=	1.50	
Factored Load		=	10.50	kN/m^2

Span Ratio l_y/l_x = 5.26/2.97

= 1.77

2. Bending Moment Calculation:

Support Condition: Two Adjacent edges discontinuous continuous)

From IS 456-2000 Table 26

αNx = 0.086

αPx = 0.065

αNy = 0.047

αPy = 0.035

Shorter Span

Negative Moment (support)

Mnx = α_{Nx} x W_ux L_x^2

= 0.086 x 10.50 x2.97^2

= 7.97kN.m

Positive Moment (Mid Span)

Mpx = α_{px} xW_u x L_x^2

= 0.065 x 10.50 x 2.97^2

= 6.02kN.m

Longer Span:

Negative Moment (Support)

MNy = α_{Ny} x W_u x L_x^2

= 0.047 x 10.50 x 2.97^2

= 4.35kN.m

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Positive Moment (Mid span) Mpy $= \alpha_{py} \times W_u \times L_x^2$

$= 0.035 \times 10.50 \times 2.97^2$

$= 3.24$kN.m

Reinforcement :

$$Mu = 0.87 \times fy \times Ast \times d \left\{ 1 - \frac{(fy \times Ast)}{fck \times bd} \right\}$$

$$= 0.87 \times 500 \times Ast \times 126 \quad 1 - \left\{ \frac{500 \times Ast}{20 \times 1000 \times 126} \right\}$$

Ast in Shorter Span

- Support $=149mm^2$
- Mid Span $=112mm^2$

Ast in Longer Span

- Support $=110mm^2$
- Mid Span $=90mm^2$

	Reinf Required		Reinf Provided
Shorter Span			
Support	8ø @ 340mm c/c	-	8 dia@ 150 mm c/c
Mid Span	8ø @ 320mm c/c	-	8 dia@ 150 mm c/c
Longer Span			
Support	8ø @ 380mm c/c	-	8 dia@ 150 mm c/c
Mid Span	8ø @ 270mm c/c	-	8 dia@ 150 mm c/c

3. Check for deflection

Allowable span / depth Ratio	l/D	=	32
% of Tension reinforcement	pt	=	0.179
Corresponding Compression reinf.	Bar dia	=	8mm
	spacing	=	150mm
% of compression reinforcement	pc	=	0.179

Modification factor for Tension reinf. F1 = 1.20 (From Fig 4 in IS 456 – 2000)

Modification factor for comp.reinf. F2 = 1.00 (From Fig 5 in IS 456 – 2000)

Modified span/ depth ratio = l/d xF1 xF2

= 32 x 1.20 x 1.00

= 38.40

Actual span / depth ratio = 2970 / 126

= 23.57

Actual span / depth ratio < Modified span /depth ratio 23.57 < 38.40 SAFE

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DESIGN OF BEAM

Data:

Effective length		=	5.11 m
		=	5110 mm
Breadth of Beam		=	230 mm
Overall depth (assumed)		=	450 mm
Characteristic strength of steel			
	fy	=	500 N/mm^2
	fck	=	20 N/mm^2
	Cover	=	25 mm

Total Load

Dead load from slab		=	3.75 kN/mm^2
Ceiling Plastering		=	0.25kN/m^2
Floor finish		=	1 kN/mm^2
Total dead load		=	5.00 kN/m^2
Live load		=	2 kN/m^2
	Total Load	=	7.00 kN/m^2

Effective Depth = d = 450 - 25 - 25 / 2

= 412.5 mm

Distance of centroid of Compression reinforcement from the extreme compression fibre =

d' = 25 +(25/2)

= 37.5 mm

$$\frac{d'}{d} = \frac{37.5}{412.5}$$

= 0.09

Minimum Reinforcement

The min.area of tension reinforcement shall not be less than the given,

$$\frac{As}{bd} = \frac{0.85}{fy}$$

Where,

As = Min.area of tension reinforcement

b = Breadth of the beam or breadth of web of T-beam

d = Effective depth

fy = Characteristic strength of reinforcement

$$As = \frac{0.85 \times b \times d}{fy}$$

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$$= \frac{0.85 \times 230 \times 412.5}{500}$$

$$= 161.29 \text{ mm2}$$

Refering Design Aids IS:456,The reinforcement as shown in the sketch

Overall depth of beam = $\frac{1}{12}$ to $\frac{1}{20}$ of the span

Provide over all depth =450 mm

Width of rib $= \frac{2}{3}(\text{depth of rib}) = \frac{2}{3}(450-150)$

=200 mm

Loading on the beam

From the Slab $= \frac{Wl_x}{6}\left(3 - \left(\frac{lx}{ly}\right)^2\right)$

$= \frac{7 \times 3.2}{6}\left(3 - \left(\frac{3.2}{5.11}\right)^2\right)$

=9.74 kN/m

D.L of the rib = 0.23 x 0.3 x 25 kN/m

= 1.8 kN/m

Total = 11.54 kN/m

Providing 16 mm dia bars at clear cover 25 mm

Effective cover = 25 + 8

= 33 mm

Effective depth d = 417 mm

Center to center bearing= 5.11 m

Effective span = 5.11 m

Max. BM $= \frac{11.54 \times 3.2 \times 3.2}{8}$

= 14.77 kN.m

Req.depth d $= \left\{\frac{1.5 \times 14.77 \times 10^6}{2.75 \times 230}\right\}^{1/2}$

= 187.16 mm

Provided depth, D = 450 mm

$\frac{Mu}{bd^2} = \left\{\frac{1.5 \times 14.77 \times 10^6}{230 \times 417 \times 417}\right\}^{1/2}$

= 0.75

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$$pt = \frac{fck}{2 \times fy}\left(1-\left\{SQRT\left[1-\frac{4 \times (Mu/b \times d^2)}{0.87 \times fck}\right]\right\}\right) \times 100$$

$= 0.18\ \%$

$$Ast = \left\{\frac{0.18 \times 230 \times 417}{100}\right\}$$

$= 2884\ mm^2$

Provided 6 – Y16

Check for Shear

$$V = \frac{1.5 \times 11.54 \times 5.11}{2}$$

$= 44.22$ kN

$T_c = 0.54\ N/mm^2$

$$V_{uc} = \frac{0.48 \times 230 \times 417}{1000}$$

$= 46.1$ kN

$V_{uc} >$ Shear at support

Provide Y8 @ 150 c / c (stirrups)

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DESIGN OF COLUMN

Data:

Total No. of floors		=	5
Total No. of Roof		=	6
Storey height		=	3 m
Breadth of column	b	=	230 m
		=	0.23 m
Depth of column	D	=	600 mm
		=	0.60 m
	fck	=	20 N/mm^2

Total axial load

Dead load & live load from one floor		=	280 kN
Total axial load all floors		=	6 x 280
		=	1680 kN
Column dimension		=	230 x 600 mm
Self weight of column		=	0.23 x 0.60 x 6 x 20 x 3
		=	50 kN
Total load		=	(1680 + 50) x 1.5
	Pu	=	2595 kN

Area of Reinforcement

$$\frac{d'}{D} = 0.08$$

$$d' = 50 \text{ mm}$$

$$\frac{Pu}{fck \times b \times D} = \frac{2595 \times 10^3}{20 \times 230 \times 600}$$

$$= 0.95$$

$$\frac{Mu}{fck \times b \times D2} = \frac{1.5 \times 30.48 \times 10^6}{20 \times 230 \times 600^2}$$

$$= 0.02$$

Refering to chart in SP : 16 for values

$$\frac{Pu}{fck.b.D} = 0.95 \quad : \quad \frac{Mu}{fck.b.D^2} = 0.02$$

The value of $\frac{P}{fck}$ = 0.05

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$$P = 0.05 \times 20$$

$$= 1.0$$

$$A_s = \frac{p \times b \times D}{100}$$

$$= \frac{1.6 \times 230 \times 600}{100}$$

$$= 2208 \text{ mm}^2$$

Provide 12 – Y16 bars.

<u>Design of Lateral Ties</u>

Dia of lateral ties $= \frac{1}{4} \times 16 = 4$ mm

Hence 8mm dia bar is adopted

Pitch is min. of the following – (as per IS: 456)

a. Least lateral dimension = 300 mm
b. 16 x dia of bar
c. 300 mm

Hence provide Y8 @ 150mm c/c
Similarly other columns are calculated.

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DESIGN OF FOUNDATION

Design of Isolated footing for Intermediate column

Data :

Column Size	L	=	230 mm
	B	=	600 mm
Size of footing	L	=	2.5 m
	B	=	2.5 m
Safe Bearing capacity of soil	SBC	=	450 kN/m^2
Depth of footing below GL		=	2 m
Density of soil	g	=	18 kN/m^3
Characteristic strength of concrete	fck	=	20 N/mm^2
Characteristic strength of Steel	fy	=	500 N/mm^2
Depth of footing provided	D	=	1.2m
Cover	d'	=	50mm

1. Loading on footing

Axial load on column = 2595 kN

Self weight of footing (10%) = 131 kN

Total load on footing = 2726 kN

Adpot size of footing = 2.5 x 2.5 m

= 6.25 m^2

Soil pressure $\frac{P}{A} = \frac{2726}{6.25}$

= 436 kN/m^2

Gross safe bearing capacity = 450 kN/m^2

Hence safe & sound

2. Moment Calculation

Face of the column $= \frac{2.5-0.60}{2}$

= 0.95 m

Design pressure = 436 kN/m^2

Moment M $= \left\{ \frac{436 \times (0.95)^2}{2} \right\}$

= 196.75 kN.m

Factored moment = 295.12 kN.m

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3. Area of Reinforcement

Effective Depth	d	=	620 mm	(or)	0.62 m
Dia of bar provided	ø	=	20 mm		
Provided spacing		=	100mm		

$$\frac{Mu}{b \times d^2} = \frac{295.12 \times 10^6}{1000 \times 620 \times 620}$$

$$= 0.77$$

Percentage of tension reinforcement

$$pt = \frac{fck}{2 \times fy}\left\{1-\left(SQRT\left[1 - \frac{4 \times (Mu/b \times d2)}{0.87 \times fck}\right]\right)\right\} \times 100$$

$$= 0.51\ \%$$

Required, $Ast = \frac{0.51 \times 1000 \times 620}{100}$

$$= 3162\ mm^2$$

Hence safe & sound

4.Check for One way shear

$$Vu = 1.5 \times 254 \times 3 \times 0.5 \times 0.30 \times 0.5 - 0.62$$

$$= 160\ kN$$

Permissible shear stress $= \frac{160 \times 1000}{1000 \times 620}$

$$= 0.26\ N/mm^2$$

Allowable shear stress $= 0.49\ N/mm^2$

Hence safe & sound

5.Check for punching shear

Punching force Pf $= \left(254 \times 608 - ((0.23 + 0.62) \times (0.60 + 0.62))\right) \times 1.5$

$$= 2187\ kN$$

Punching shear stress $= \frac{2,187 \times 1000}{620 \times 2 \times (230 + 6000 + (2 \times 620))}$

$$= 0.85\ N/mm^2$$

Allowable shear stress $= 1.25\ N/mm^2$

Hence safe & sound

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EARTH QUAKE ANALYSIS

1. Calculation of horizontal seismic Coefficient

As per IS 1893 – 2002

$$Ah = \frac{z \times 1 \times Sa}{2 \times R \times g}$$

Zone factor	Z	=	0.16	(As per table 7)
Importance factor	I	=	1	
Response reduction factor,	R	=	5	

$$\frac{Sa}{g} = 2.5$$

Natural period, Ta = $0.075\ h^{3/4}$

(Clause 7.6) h = height of the building

= $0.075 \times 13.1^{0.75}$

= 0.52 sec

$$Ah = \frac{0.16 \times 1 \times 2.5}{2 \times 5}$$

= 0.04

2. Calculation of Base shear & seismic forces

Base shear V = $Ah \sum Wi$

= 0.040 x 1316

= 52.64 kN

Total no of columns = 21

Shear per column = $\frac{25.64}{21}$

Seismic moment per column = $\frac{1.22 \times 3}{4}$

= 0.92 T-m

Column provided = 230 x 600 mm with 12 –Y16

Hence safe & sound

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3. Checking for footing

SBC = $45\ T/m^2$

Soil pressure = $25.4\ T/m^2$

$$\frac{P}{A} \pm \frac{1.40 \times 6}{12}$$

$= 25.4 \pm 0.7$

$= 26.1$

$= 24.7\ T/m^2 < 40.00\ T/m^2$

Hence safe & sound

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