**Design of column**

**Design of loads:-**

Load from rectangular beam = 8.69 x 2.7/2

 = 11.73Kn

Load from rectangular beam = 7.89 x 2.5/2

 = 9.86Kn

Load from rectangular beam = 11.965x 3.8/2

 = 22.73Kn

Load from rectangular beam = 10.49 x 3.3/2

 = 17.31Kn

 Total Load = 61.63

 Design Load = 61.63 x 92.45KN

**Check for stiffness:-**

Two end fixed off length of the column lef

 = 0.65 x L

 = 0.65 x 3800

 = 2470mm

 Slenderness ratio = 2470/230

 = 10.73 < 12

Hence the column is a short column

**Size of Column:-**

Let the gross area of the column = Ag

Assume 1% longitudinal steel in column

Area of steel reinforcement Asc = 0.01Ag

Area of Constant Ac = Ag – 0.01Ag

 = 0.99Ag

**Substituting in the strength equations:-**

0.4 x fck x Ac + 0.67 x fy x Asc

0.4 x 20x 0.99 Ag + 0.67 x 415 x 0.01Ag = 92.45 x 103

 Ag = 92.45 x 103/10.701

 = 8639.37mm2

Size of square column = $\sqrt{8639.37}$

 = 92.94 $≈$ 230mm

Longitudinal Reinforcement:-

Area of steel required = 0.01 x 8639.37

 = 86.39mm2

Minimum diameter lateral is greats of

1. 1/4 x $ø$ of longitudinal bar = 1/4 x 12 = 3mm

2. 3mm Say = 6mm

Maximum permitted pitch is the least of

1. Least of internal diameter of column
2. 16 x diameter of longitudinal bar = 16 X6 = 96mm
3. 300mm Say 8mm ø later ties @

**Design of Plinth Beam**

**Size of Beam:-**  = 230 x 300mm

**Loads:-**

 Self weight of plinth beam = 0.23 x 0.3 x 1 x 25

 = 1.725kn

 Load on the wall = 0.23 x 3 x 1 x 20

 = 13.8KN/m

 Total load = 15.53KN/m

 Design load = 15.53 x 1.5

 = 23.29KN/m

**Bending moment:-**

Bending moment (B.M) = wl2/8

 = 23.29 x 3.8/8

 = 11.06KN.m

 B.M = 11.06 X103

**Depth required:-**

 M20 grade concrete fe415 grade steel

I.S. 456 = 2000

 Mu = 2.76 b d2

 d = $\sqrt{11.06 x 10\^2/2.76x230}$

 = 131.99mm $≈$ 132mm

132 < 300mm

Hence safe

**Area of steel:-**

 Mu = 0.87 x fy x Ast( d- fy x Ast/ fck x b)

11.06 x 106 = 0.87 x 415 x Ast x (132-415 x Ast/20 x 230)

11.06 x 106 = 361.05 Ast(132-0.09 Ast)

11.06 x 106 = 47658.6 Ast  - 32.56 Ast2

 Ast = 289.21mm2

Minimum area of steel = 0.85 x b x d/ fy

 = 0.85 x 230 x 300/415

 = 141.32mm2

Maximum area of steel = 0.04 x b x d

 = 0.04 x 230 x 300

 = 2760mm2

**Column Footing Design**

Size of column = 230 x 230mm

Column load = 61.63KN

Plinth beam load = 15.5KN

M20, fe415

Capacity of soil = 150KN/m2

Assume self weight of column 10%

Self weight of column = 77.16 x 10/100

 = 7.716KN

Total load = 84.876KN

Design load = 84.876 x 1.5

 = 127.314KN

Area = 127.314 / 150

 = 0.848m2

 a2 $=>$ a = $\sqrt{0.848}$

 = 0.920 $≈$ 0.9m

1. **Size of footing:-**

 0.9 x .09

 Original area = 0.81m2

1. **Upward pressure:-**

 W = column load x f.o.s /area

 = 77.16 x 1.5 / 0.81

 = 142.88KN/m2

**Bending Moment:-**

 Mu = w x 0.355 x 0.335/2

 = 142.88 x0.9 x0.355 x0.335/2

 = 7.215N.mm

 Mu = Qu x b x d2

 d = $\sqrt{7.215 x10^{6}/2.78x900}$

 = 53.70mm $≈$ 50mm

**Depth required:-**

 = 50 + 50

 d = 100mm

 say = 300mm

**Overall depth:-**

 D = d + clear cover + dia/2

 = 300 + 40 + 6

 = 346mm

 Mu limit = Qu x b x d2

 = 2.76 x 900 x (300)2

 = 223.56 x $10^{6}$ N.mm

 Mu = Mu limit

 Under reinforced section

 Mu = 0.87 x fy x Ast ( d- fy x Ast/ fck x b)

7.215 x 106 = 0.87 x 415 x Ast (300-(415 x Ast/20 x 900))

7.215 x 106 = 361.05 Ast (3000.02305 Ast)

7.215 x 106 = 108.31 x 103 Ast – 8.32 Ast2

 8.32 Ast2 - 108.31 x 103 Ast + 7.215 x 106

 Ast = 66.11mm2

**Ast min**= 0.12% of b x D

 = 0.12/100 x 900 x 346

 = 373.68mm2 $≈$ 374mm2

**Spacing:-**

Π/4 x d2/ Ast x b = Π/4 x 122/ 374x 900

 = 272.15mm

**No`s of rod:-**

 = Ast/ Π/4 x d2

 = 374/ Π/4x(122) = 3.306 No`s $≈$ 3 No`s

**Bending moment:-**

 Projection of footing from face of column = (0.9-0.23/2)

 = 0.335

 $τ$c=100 x Ast/bxd

 =100x374/900x300

 = 0.28N/mm2

Critical section yy for transverse shear is at distance of 300mm from face of column and 40mm free edge shear force yy

Vy=net upward x bx(projection –b)

 = 131.77x0.9x(0.335-0.3)

 =4.15 KN(or)4150.755N

Total depth of footing at the middle of section xy

 =300+((346-300)/340x40)

 =300.003mm

Eff.depth of cente:- =300.003-75

 =225.003mm

Eff.depth of edge = 300-75

 =225mm

Breath of section at top =230x(2x300)

 =830mm

Eff.are the transverse shear

 =900x225+(830+900/2)x150

 =332.25x103mm2

**Nominal shear stress:-**

 τvy =4150.755/332.25 x103

 =0.012 N/mm2

 0.012 N/mm2<0.28 N/mm2

 Hence ok

Cheek for SBC

load (p) =77.16KN

self load=l x b x D x unit weight of concrete

 =0.9x0.9x.346x25

 =7.006Kn

Total load =77.16+7.006

 =84.46Kn

Self bearing capacity(sbc) =total load/area =84.16/0.9x0.9 =103.908KN/m2

 Given SBC =150 KN/m2

 SBC (provided) = 103.908KN/m2

 Given SBC<SBC provided

 Hence ok

Lintel cum sunshade over window size design of sunshade

Assume an uniform thickness of 60mm with an effective depth=42

Loads

Self weight of sunshade =1x0.6x0.6x25

 =0.9kN

Imposed load =1x0.6x0.25

 =0.15Kn

Total load =1.05kn

Design load =1.05x1.5=1.575n/M

**Reinforcement**

Mu wu l2 / 8=1.575 x 12

 =0.1968KNm (or) 196.875 x 102

196.875 x 102 =0.87 x 415 x Ast (42-(415 x Ast20 x 1000))

196.875 x 102 =361.05 Ast(42-0.02075)

196.875 x 102 =15164.1 Ast -7.49 Ast2

 7.49 Ast2 - 15164.1 Ast + 196.875 x 102

 Ast =13.06mm2

 Ast minimum = 0.12/100x(1000x42)

 = 50.4mm2

Spacing of 6 mm ø bar =28x1000x42

 =555.55mm c/c

**Ast provided**

 = Π/4 x d2xno of bar

 Ast = Π/4x62x3

 = 84.82mm2

**check for shear**

vu=wx1.5=1.05x1.5=1.575kn

nominal shear stress $τ$v= vu/bd

 = 1.525x103/1000x42

 =0.0375n.mm2

 K = 1.3

 Ast = π/4 x d2/2 x 555.55 x 1000

 = 25.44mm2

 %Ast = Ast/bd x 100

 = 25.44/1000 x 42 x 100

 = 0.0605%

 $τ$c for M20 concrete = 0.28 N/mm2

 K $τ$c  = 1.3 x 0.28

 = 0.364N/mm2

 $τ$c max/2 = 2.8/2 = 1.4N/mm2

 $τ$v < K $τ$c < $τ$c max/2

 Safe in shear

**Check for defelection:-**

 d = 621/7 x 0.19

 = 95

 fs = 0.58 x 415 x 50.4/84.82

 = 143.03 $≈$ 145

 %Ast = Ast/bd x 100

 = 50.4/100 x 422

 = 0.12

 Mf = 2.8

 d = 621/7 x 2.8

 = 31.68mm

 d(available) < d(provide)

 Design is safe

 **Design of lintel over door**

Assume the size of lintel as = 230 x 150mm

Width of lintel = thickness of wall

Assume an uniform thickness of 60mm width an Eff depth

 = 42mm

**Loads:-**

Self weight of sunshade = 1 x 0.6 x 0.06 x 25

 = 0.9KN

Imposed load = 1 x 0.6 x 25

 = 15KN

Total load = 15.9KN

Design load = 15.9 x 1.5

 = 23.85 x KN/m

**Bending Moment:-**

B.M @ Support = 23.85x0.6/2

 = 7.155KN.m

 (or)

 = 7.155x106N-mm

**Area of steel:-**

 Mu = 0.87 x fy x Ast ( d- fy x Ast/ fck x b)

7.155 x 106 = 0.87 x 415 x Ast (42-(415 x Ast/20 x 230))

7.155 x 106 = 361.05 Ast (42-0.090 Ast)

7.155 x 106 = 15.1641 x 103 Ast – 32.49 Ast2

 32.49Ast2 - 15.1641 x 103 Ast + 7.155 x 106

 Ast = 233.33mm2

**Ast minimuim** = 0.12 / 100 x (1000 x 42)

 = 50.4 mm2

Spacing 6mm$ ø $bar = Π/4 x d2/ Ast x 230

 = 27.87mm c/c

Provide 1 No’s of 6mm $ø$ bar distributions @ 27.8787mm c/c

**Design of lintel:-**

Size of lintel = 230 x 150

Using 6mm $∅$bar with nominal cover of 20mm

**Eff depth** = 150 – 20 – 6/2

 = 127mm

**Eff span:-**

A . c/c distance bearing = door size + 0.15

 = 1.2 + 0.15

 = 1.35m

B . Clear opening + eff depth = 1.2+ 0.127

 = 1.327m

Eff depth of l = 1.327m

Height of wall above the lintel = 0.9 – 0.15

 = 0.75 <1.327

Above the lintel:-

Weight masonry = 1.327 x 0.75 x 0.23 x 20

 = 4.578KN

Self weight of lintel = 0.23 x 0.15 x 1.327 x 20

 = 0.916KN

Total load = 5.494KN

Design load = 5.494 x 1.5

 = 8.241KN/m2

**Bending moment:-**

 Maximum B.M = 8.241 x 1.327/8

 = 1.13669KN.m

**Depth required:-**

Eff depth required = √1.3669 x 10^6/(2.76 x 230)

 = 46.40mm < 127mm

 Hence okays

**Area of steel:-**

 Mu = 0.87 x fy x Ast (d- fy x Ast/fck x b)

1.3669 x 10^6 = 0.87 x 415 x Ast(127-415 x Ast/20 x 230)

1.3669 x 10^6 = 361.05 Ast(127-0.090 Ast)

1.3669 x 10^6 = 361.05 Ast -32.49 Ast

Providing minimum 2 no`s of 6mm dia bar booth 6mm dia top of hanger bar