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Proj. Std. file name :	XXXXXXXX INDUSTRIES	Unit / Equipment No.	Sub-Stations No.2
Prep. By / Chkd. By :	VV/JA	Design Status :	RA

Title : DESIGN OF R.C.C. COLUMN WITH AXIAL FORCE, BIAXIAL MOMENTS, i.e. ABOUT X AND Z DIR & DESIGN OF ISOLATED FOOTING (OPTIONAL)

(APPLICABLE SOFTWARE : MICROSOFT EXCEL)
(LIMIT STATE DESIGN AS PER IS:456-1978)

1.0 DEFAULT VALUES

1.1	Material Properties :					
1.1.1	Grade of Conc:(M 15/20/25/30/35/40)	M 30	fck	=	30	N/mm²
1.1.2	Grade of steel:(Fe 250/415/500)	Fe 500	fy	=	500	N/mm²
1.2	Unit weight of concrete			=	2.5	T/m³
1.3	Cover to reinforcement in Z- Direction			=	45.00	mm
1.4	Cover to reinforcement in X- Direction			=	45.00	mm

2.0 INPUT DATA

2.1 GENERAL DATA

2.1.1	Unit	:	FTG-F1
2.1.2	Joint no (As per computer output)	:	14
2.1.3	Column no.	:	C13
2.1.4	Reference drawing no.	:	Dwg: 6469-CI-UGI
2.1.5	Civil Design Basis No.	:	
2.1.6	Remarks	:	**FOR LC.-101,122,**

2.2 DIMENSIONS OF COLUMN

2.2.1	Longer dimension of column along X axis.	D	=	400	mm
2.2.2	Shorter dimension of column along Z axis.	b	=	400	mm
2.2.3	Unsupported length for bending parallel to larger dimension	Lx	=	3.00	m
2.2.4	Unsupported length for bending parallel to shorter dimension.	Lz	=	3.00	m

(Unsupported length as per clause 24.1.3 of IS:456-1978)

2.2.5	Multi. Factor for calculating eff. Length for bending about Z axis.	rx	=	1.5	
2.2.6	Multi. Factor for calculating eff. Length for bending about X axis.	rz	=	1.5	

Unsupported length multiplication factor as per Table 24, pg 144 of IS:456 -1978 for calculation of effective length depending on end restraint conditions.

Effective length of bending parallel to larger dimension. { Lx * rx }	Lex	=	4.50	m
Effective length of bending parallel to shorter dimension. { Lz * rz }	Lez	=	4.50	m

2.3 LOADS ON COLUMN

2.3.1 LOAD CASES CONSIDERED :

1. (DL + LL.) / DL
2. (DL + LL.) / DL + Wind Load / Earthquake Load in X-Direction
3. (DL + LL.) / DL + Wind Load / Earthquake Load in Z-Direction

Based on the Load Combination in TABLE 12 of IS:456-1978

Partial safety factor for Load case 1 is taken as 1.5

Input the partial safety factor for Load case 2 (i.e. 1.2 or 1.5)	1.2
Input the partial safety factor for Load case 3 (i.e. 1.2 or 1.5)	1.2

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2.3.2 Load case 1. DL. + LL./ DL.

Axial load on compression member inclusive of selfweight.	P	=	45.20	T
Moment in the direction of larger dimension (i.e. about Z axis)	Mz	=	1.10	T-M
Moment in the direction of shorter dimension (ie. about X axis)	Mx	=	1.10	T-M
Load case 2. DL. + LL. + WL/EQL in X-direction				
Axial load on compression member inclusive of selfweight.	P	=	66.20	T
Moment in the direction of larger dimension (ie. about Z axis)	Mz	=	11.60	T-M
Moment in the direction of shorter dimension (i.e. about X axis)	Mx	=	1.10	T-M
Load case 3. DL. + LL. + WL/EQL in Z-direction				
Axial load on compression member inclusive of selfweight.	P	=	51.50	T
Moment in the direction of larger dimension (i.e. about Z axis)	Mz	=	1.10	T-M
Moment in the direction of shorter dimension (i.e. about X axis)	Mx	=	9.50	T-M

2.4 REINFORCEMENT

2.4.1 POSITIONING OF REINFORCEMENT

Case1. Reinforcement only along shorter direction.
Case2. Reinforcement only along longer direction.
Case3. Reinforcement equally distributed on four faces.

Input case under consideration : = **3**

2.4.2 Properties of reinforcement :

Diameter of lateral ties		=	8	mm
Diameter of bar provided	d	=	16	mm.
No. of bars provided.	N	=	12	No.
% of reinforcement provided.	Ptpro	=	1.509	

' VALUE OF Pt IS WITHIN LIMITS OF MAX. & MIN. Pt..'

2.4.3

Effective cover to main reinforcement in Z-Dir.	d1	=	53.00	mm
Effective cover to main reinforcement in X-Dir.	d2	=	53.00	mm

3.0 CALCULATIONS (For Load Case 1)

3.1 MOMENTS DUE TO SLENDERNESS.

3.1.1 Lex/D = 11.25

<12.HENCE NOT SLENDER

Lez/b = 11.25

<12.HENCE NOT SLENDER

3.1.2 ADDITIONAL MOMENTS DUE TO SLENDERNESS.

Additional eccentricity for slender compression member in X-Direction

{ D*(Lex/D)²/2000 }	eax	=	0.00	mm
Additional moment in X-Direction { P * eax }	Maz	=	0.00	T-m

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Additional eccentricity for slender compression member in Z-Direction
{ b*(Lez/b)²/2000 } — eaz = 0.00 mm
Additional moment in Z-Direction { P * eaz } — Max = 0.00 T-m
Above moments will have to be reduced in accordance
with clause 38.7.1.1 of IS:456-1978.

3.1.3 Maxm axial Load on the compression member.{0.45fck Ac+0.75fyAs} — Puz = 309.13 T

3.1.4 Calculation of Pb for rectangular section
From table 60 of SP-16
For rectangular section and d2/D ratio = 0.1325 — k1z = 0.196
For rect. sec. & equal reinf. on all sides and d2/D same as above — k2z = 0.256
Pbz(about zz-axis) = (k1z + k2z * Ptpro /fck) * fck * b * D — Pbz = 102.20 T
For rectangular section and d1/b ratio = 0.1325 — k1x = 0.196
For rect. sec. & equal reinf. on all sides and d1/b same as above — k2x = 0.256
Pbx(about xx-axis) = (k1x + k2x * Ptpro / fck) * fck * b * D — Pbx = 102.20 T

3.1.5 Calculation of Reduction coefficients.
Reduction coefficient for Maz = { (Puz - Pu) / (Puz - Pbz) } — Kz = 1.000
Reduction coefficient for Max = { (Puz - Pu) / (Puz - Pbx) } — Kx = 1.000
If Kx / Kz >1 or < 0 , then Kx or Kz = 1

3.1.6 Additional moments after accounting for reduction coefficients.
Additional moments about Z-axis {Maz * kz} — Maz1 = 0.00 T-m
Additional moments about X-axis {Max * kx} — Max1 = 0.00 T-m

3.2 MOMENTS DUE TO MINIMUM ECCENTRICITY
Minimum eccentricity in X-Dir. { Max [(Lx/500 + D/30) , 20mm] } — ex = 20.00 mm
Moment due to minimum eccentricity in X-dir. {P* ex } — Mzmin = 0.90 T-m
Minimum eccentricity in Z-Dir. = { Max [(Lz/500 + b/30) , 20mm] } — ez = 20.00 mm
Moment due to minimum eccentricity in Z-dir. {P* ez } — Mxmin = 0.90 T-m

3.3 DESIGN MOMENTS AND DESIGN AXIAL LOAD FOR WHICH THE COLUMN IS DESIGNED :

Load case 1. DL. + LL./ DL.
{ [(Max (Mz , Mzmin)) + Maz1] * 1.5 } — Muz = 1.65 T-m
{ [(Max (Mx , Mxmin)) + Max1] * 1.5 } — Mux = 1.65 T-m
{ P * 1.5 } — Pu = 67.80 T

Load case 2. DL. + LL. + WL/EQL in X-direction
{ [{ Max (Mz , Mzmin) } + Maz1] * 1.2 } — Muz = 13.92 T-m
{ [{ Max (Mx , Mxmin) } + Max1] * 1.2 } — Mux = 1.59 T-m
{ P * 1.2 } — Pu = 79.44 T

Load case 3. DL. + LL. + WL/EQL in Z-direction
{ [{ Max (Mz , Mzmin) } + Maz1] * 1.2 } — Muz = 1.32 T-m
{ [{ Max (Mx , Mxmin) } + Max1] * 1.2 } — Mux = 11.40 T-m
{ P * 1.2 } — Pu = 61.80 T

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3.4 INTERACTION CHECK FOR MEMBERS SUBJECTED TO AXIAL FORCE AND BIAXIAL BENDING.

Ptpro/fck = 0.0503 %

Load case 1. DL. + LL./ DL.

Pu/fck * b * D = 0.139

Muz1/(fck*b*D²) = 0.104

Maximum uniaxial moment capacity about z-axis for an axial load of Pu. Muz1 = 20.27 T-m

Mux1/(fck*b²*D) = 0.104

Maximum uniaxial moment capacity about x-axis for an axial load of Pu . Mux1 = 20.31 T-m

Load case 2. DL. + LL. + WL/EQL in X-direction

Pu/fck * b * D = 0.16236

Muz1/(fck*b*D²) = 0.104

Maximum uniaxial moment capacity about z-axis for an axial load of Pu. Muz1 = 20.44 T-m

Mux1/(fck*b²*D) = 0.102

Maximum uniaxial moment capacity about x-axis for an axial load of Pu . Mux1 = 20.01 T-m

Load case 3. DL. + LL. + WL/EQL in Z-direction

Pu/fck * b * D = 0.12630

Muz1/(fck*b*D²) = 0.103

Maximum uniaxial moment capacity about z-axis for an axial load of Pu. Muz1 = 20.18 T-m

Mux1/(fck*b²*D) = 0.102

Maximum uniaxial moment capacity about x-axis for an axial load of Pu . Mux1 = 20.01 T-m

Calculation of αn :

if Pu/Puz < 0.2 then αn = 1.0

if Pu/Puz > 0.8 then αn = 2.0

else αn =(1.0+((Pu/Puz)-0.2)/0.6) = = 1.03

$[\ Mux / Mux1\ ]^{\alpha n} + [\ Muz / Muz1\ ]^{\alpha n}$ = 0.150 <1

' HENCE, THE SECTION SELECTED IS SAFE FOR LOAD CASE I. '

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3.5 Similarly interaction check is given for other load cases ,the calculations of which are tabulated as follows :

	LOAD CASES		
	1	2	3
P (T)	45.200	66.200	51.500
Mz (T-m)	1.100	11.600	1.100
Mx (T-m)	1.100	1.100	9.500
Load factor	1.500	1.200	1.2000
Kz	1.000	1.000	1.000
Kx	1.000	1.000	1.000
Maz1	0.000	0.000	0.000
Max1	0.000	0.000	0.000
Pu (T)	67.800	79.440	61.800
Muz (T-m)	1.650	13.920	1.320
Mux (T-m)	1.650	1.589	11.400
Pu/fck*b*D	0.139	0.162	0.126
Muz1/fck*b*D²	0.104	0.104	0.103
Muz1 (T-m)	20.270	20.442	20.181
Mux1/fck*b²*D	0.104	0.102	0.102
Mux1 (T-m)	20.308	20.006	20.006
Pu/Puz	0.219	0.257	0.200
αn	1.032	1.095	1.000
Interaction check	0.150	0.719	0.635
	<1 ..... SAFE	<1 ..... SAFE	<1 ..... SAFE

-----------------------------HENCE SECTION IS SAFE

3.6 LATERAL TIES

3.6.1 Spacing of the lateral ties = {Min (48*dia. of stps, 16*dia. of main rein., ' b')} = 250 mm

Cl. 25.5.3.2 c of IS:456-1978.

Distance between two bars along longer dimension (D) = 98.00 mm

Distance between two bars along shorter dimension (b) = 98.00 mm

PROVIDE A SINGLE OPEN LOOP TO TIE THE REINFORCEMENT IN LONGER DIRECTION

PROVIDE A SINGLE OPEN LOOP TO TIE THE REINFORCEMENT IN SHORTER DIRECTION

SUMMARY OF COLUMN DESIGN.

Longer dimension of column (D)	=	400	mm
Shorter dimension of column (b)	=	400	mm
Diameter of vertical main bars	=	16	mm
No.of bars	=	12	nos.
Diameter of lateral ties	=	8	mm
Spacing of lateral ties	=	250	mm

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PART B :- DESIGN OF ISOLATED RECTANGULAR FOOTING (OPTIONAL).

4.0 DEFAULT DATA FOR FOOTING :

Clear cover at bottom for footing		=	75	mm
Clear cover at sides and top for footing		=	50	mm
Coefficient of friction against sliding	μ	=	0.4	
Factor of safety against sliding (As per Cl. 19.2 of IS:456-1978)	FOSs	=	1.5	
Factor of safety against overturning (As per Cl. 19.1 of IS:456-1978)	FOSo	=	1.5	
Density of soil	ds	=	1.70	T/m³

5.0 INPUT DATA FOR FOOTING :

Depth of bottom of footing below FFL	h'	=	3960	mm
Length of the footing along X-axis	Fx	=	1900	mm
Length of the footing along Z-axis	Fz	=	4500	mm
Thickness of footing	Tf	=	600	mm
Net SBC of soil at bottom of footing		=	12.00	T/m²

REINFORCEMENT

Bar dia. to be used along X-axis @ bottom of footing. (min 12mm)	=	12	mm
Bar dia. to be used along Z-axis @ bottom of footing. (min 12mm)	=	12	mm
Bar dia. to be used along Z-axis and X-axis @ top of footing. (min 10mm)	=	10	mm

6.0 CALCULATIONS FOR FOOTING

Plan area of footing = { Fx * Fz }	Af	=	8.55	m²
Section modulus of footing for bending about X-Axis = {(Fx * Fz²)/6}	Zx	=	6.41	m³
Section modulus of footing for bending about Z-Axis = {(Fz * Fx²)/6}	Zz	=	2.71	m³
Selfweight of the footing = {Af * Tf * density }		=	12.83	T
DL. of soil { (Af - (D*b)) * (h'-Tf) * ds }		=	47.92	T

6.1 DESIGN OF ISOLATED RECTANGULAR R.C.C. FOOTING

(Subjected to axial load and Biaxial bending)

Net Pressure intensities and gross pressure intensities for individual load cases are calculated as follows :

Max. net pressure = (P+Wt. of footing) /Af + Mx/Zx + Mz/Zz

Min. net pressure = (P+Wt. of footing) /Af - Mx/Zx - Mz/Zz

Max. gross pressure = (P+Wt. of soil +Wt. of footing) / Af + Mx/Zx + Mz/Zz

Min. gross pressure = (P+Wt. of soil +Wt. of footing) / Af - Mx/Zx - Mz/Zz

Gross SBC of soil = { Net S.B.C. + h'*ds } = 18.732 T/m²

% Increase in SBC (Cl. 8 of IS:875(part 5)for wind & Cl. 3.3.3 of IS:1893 for seismic) = 25%

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Table showing loads and pressures for finding out critical loads on Footing

	Load case		
	1	2	3
Pmax (T)	45.20	66.20	51.50
Mx @ bottom of footing (T-m)	1.10	1.10	9.50
Mz @ bottom of footing (T-m)	1.10	11.60	1.10
Load factor	1.5	1.2	1.2
Net press.@ bot. of footing (T/m²)			
Max.	7.36	13.70	9.41
Check for Maxm. net pressures.	SAFE	SAFE	SAFE
Min.	6.21	4.79	5.64
Check for Minm. net pressures.	SAFE	SAFE	SAFE
Gross press.@ bot.of footing(T/m²)			
Max.	12.97	19.30	15.02
Check for Maxm. gross pressures.	SAFE	SAFE	SAFE
Min.	11.81	10.39	11.24
Check for Minm. gross pressures.	SAFE	SAFE	SAFE

6.1.1 **Values to be considered for Design of footing**

Note : User to input load case (Exactly as shown above):

1. Critical load case = 1
2. Critical axial load P = 45.200 T
3. Moment about X-axis = 1.100 T-m
4. Moment about Z-axis = 1.100 T-m
5. Load factor assumed (L.F.) = 1.500

6.1.2 **Development Lengths required in Tension and compression.**

6.1.2.1 Check for depth of footing as per Development length required in compression

The development length required in compression is calculated on the basis of the actual stress in the bars as the bars are not stressed fully.

Min. depth of the footing req.

{ Ld=0.87 x fy x dia. / (4x τbd x 1.25) x Max ((Muz / Muz1), (Mux / Mux1)) } = 400 mm

DEPTH OF FOOTING IS SUFFICIENT

6.1.2.2 Development length required in tension. = 500 mm

{ Ld=0.87xfy x dia./ (4xtbd) x Max ((Muz / Muz1), (Mux / Mux1)) }

6.1.3 **BEARING PRESSURES**

MAXIMUM NET PRESSURE INTENSITY LESS THAN NET S.B.C. ----------SAFE

MAXIMUM GROSS PRESSURE INTENSITY LESS THAN GROSS S.B.C. ----------SAFE

MINIMUM GROSS PRESSURE INTENSITY GREATER THAN ZERO----------SAFE

6.1.4 **Design loads and pressures**

Design forces (factored) :

Pu = {P + DL. of footing + DL of soil} * Load factor — Pu = 158.923 T

Mux = { Mx * Load factor } — Mux = 1.650 T-m

Muz = { Mz * Load factor } — Muz = 1.650 T-m

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Gross Bearing Pressure values :

qP = { Pu/A + Mux/Zx + Muz/Zz } =	qP	=	19.454 T/m²
qS = { Pu/A + Mux/Zx - Muz/Zz } =	qS	=	18.235 T/m²
qQ = { Pu/A - Mux/Zx + Muz/Zz } =	qQ	=	18.940 T/m²
qR = { Pu/A - Mux/Zx - Muz/Zz } =	qR	=	17.721 T/m²

Net Factored Loads (excluding wt. of soil) :

Pu' = {P+wt. of footing} * Load factor =	Pu'	=	87.038 T

Net Bearing Pressure values are :

qP'= { Pu'/A + Mux/Zx + Muz/Zz } =	qP'	=	11.047 T/m²
qS'= { Pu'/A + Mux/Zx - Muz/Zz } =	qS'	=	9.828 T/m²
qQ'= { Pu'/A - Mux/Zx + Muz/Zz } =	qQ'	=	10.532 T/m²
qR'= { Pu'/A - Mux/Zx - Muz/Zz } =	qR'	=	9.313 T/m²

6.1.5 DESIGN OF FOOTING

6.1.5.1 Bending Moment about X-direction at bottom of footing

Dist. of edge of footing from face of column along Z-dir. { (Fz-b)/2 }	dz	=	2.050 m
Pressure ordinate on S-R side at edge of footing (Corr. to face of column) {[(qS-qR)/Fz]*[Fz-dz] + qR}	X1	=	18.001 T/m²
Pressure ordinate on P-Q side at edge of footing (Corr. to face of column) {[(qP-qQ)/Fz]*[Fz-dz] + qQ}	X2	=	19.220 T/m²
Average pressure at the face of column on Z-axis = { (X1+X2)/2 }	Avpx1	=	18.610 T/m²
Average pressure at the edge of footing on Z-axis = { (qS+qP)/2 }	Avfx1	=	18.845 T/m²
Max. Bending moment at the face of column, Mpx1 {(Avpx1+2Avfx1)*dz²/6-L.F.*[(h'-Tf)*ds+Tf*density]*dz²/2}		=	16.702 T-m/m

6.1.5.2 Bending Moment about Z-direction at bottom of footing

Dist. of edge of footing from face of column along X-dir. { (Fx-D)/2 }	dx	=	0.750 m
Pressure ordinate on P-S side at edge of footing (Corr. to face of column) {[(qP-qS)/Fx]*[Fx-dx] + qS}	X3	=	18.973 T/m²
Pressure ordinate on Q-R side at edge of footing (Corr. to face of column) {[(qQ-qR)/Fx]*[Fx-dx] + qR}	X4	=	18.458 T/m²
Average pressure at the face of column on X-axis = { (X3+X4)/2 }	Avpz1	=	18.716 T/m²
Average pressure at the edge of footing on X-axis = { (qP+qQ)/2 }	Avfz1	=	19.197 T/m²
Max. Bending moment at the face of column, Mpz1 {(Avpz1+2*Avfz1)*dx²/6-L.F.*[(h'-Tf)*ds+Tf*density]*dx²/2}		=	2.311 T-m/m

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6.1.5.3 **Bending Moment & Steel required at bottom**

		@ X-Axis (Along Fz)	@ Z-Axis (Along Fx)
1.	Bending Moment (T-m / m)	16.702	2.311
2.	Effective depth required (mm)	202.644	75.386
	{ d = [(Mu/Ru*b] ½ }		
	where Ru=fck x 0.133		
3.	Effective depth available (mm)	519	507
		HENCE SAFE.	**HENCE SAFE.**
4.	Ast. Reqd.(mm²) {Per Metre length}	743.877	103.217
5.	Min Ast. Reqd.(mm²) Per Metre length}	720	720
	{0.12% of 1000*Tf}		
6.	Ast to be provided {Per Metre length}	743.877	720.000

Provide Bars :	**12 mm dia @**	**12 mm dia @**
@Bottom	**150 mm c/c**	**150 mm c/c**
	along Fz	**along Fx**

No. of bars reqd.	13 Nos.	31 Nos.
Actual Ast Provided	754 mm²	754 mm²
pt provided =	0.145 %	0.149 %

6.1.5.4 **Bending Moment about X-direction at top of footing**

Pressure ordinate on S-R side at edge of footing (Corr. to face of column)
{[(qS-qR)/Fz]*dz + qR} X5 = 17.955 T/m²
Pressure ordinate on P-Q side at edge of footing (Corr. to face of column)
{[(qP-qQ)/Fz]*dz + qQ} X6 = 19.174 T/m²
Average pressure at the face of column on Z-axis = { (X5+X6)/2 } Avpx2 = 18.565 T/m²
Average pressure at the edge of footing on Z-axis = { (qR+qQ)/2 } Avfx2 = 18.330 T/m²
Max. Bending moment at the face of column, Mpx2
{(Avpx2+2*Avfx2)*dz²/6-[(h'-Tf)*ds+Tf*density]*dz²/2*L.F} = 15.949 T-m/m

NO TENSION IS DEVELOPED AT THE TOP OF FOOTING.
HENCE PROVIDE MINIMUM REINFORCEMENT.

6.1.5.5 **Bending Moment about Z-direction at top of footing**

Pressure ordinate on P-S side at edge of footing (Corr. to face of column)
{[(qP-qS)/Fx]*dx + qS} X7 = 18.716 T/m²
Pressure ordinate on Q-R side at edge of footing (Corr. to face of column)
{[(qQ-qR)/Fx]*dx + qR} X8 = 18.202 T/m²
Average pressure at the face of column on X-axis = { (X7+X8)/2 } Avpz2 = 18.459 T/m²
Average pressure at the edge of footing on X-axis = { (qS+qR)/2 } Avfz2 = 17.978 T/m²
Max. Bending moment at the face of column, Mpz2 = 2.059 T-m/m
{(Avpz2+2Avfz2)*dx²/6-[(h'-Tf)*ds+Tf*density]*dx²/2*L.F}

NO TENSION IS DEVELOPED AT THE TOP OF FOOTING.
HENCE PROVIDE MINIMUM REINFORCEMENT.

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Prep. By / Chkd. By :	VV/JA	Design Status :	RA

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		@ X-Axis (Along Fz)	@ Z-Axis (Along Fx)
1.	Bending Moment (T-m / m)	15.949	2.059
2.	Effective depth required (mm)	198.024	71.148
	{ d = [(Mu/Ru*b] ^{½} }		
	where Ru=fck x 0.133		
3.	Effective depth available (mm)	545	535
		HENCE SAFE.	HENCE SAFE.
4.	Ast. Reqd.(mm²) {Per Metre length}	0.000	0.000
5.	Min Ast. Reqd.(mm²) Per Metre length}	360	360
	{0.06% of b*Tf}		
6.	Ast to be provided {Per Metre length}	360.000	360.000

Provide Bars : @Top	**10 mm dia @ 210 mm c/c along Fz**	**10 mm dia @ 210 mm c/c along Fx**

No. of bars reqd.	10 Nos.	22 Nos.
Actual Ast Provided	374 mm²	374 mm²
pt provided =	0.069 %	0.070 %

6.1.5.6 **Check for Shear:(Note:Net bearing press. values are considered for stress computation.)**

6.1.5.6.1 **One way Shear**

(Note: Respective effective depths have been considered in each direction.)

Parallel to Z-direction :

Dist. of critical section from edge of footing along Z-dir. {dz-d}	d1	=	1.531 m
Pressure ordinate corr. to critical section on S-R side {(qS'-qR') / Fz * (Fz-d1) + qR'}	X9	=	9.653 T/m²
Pressure ordinate corr. to critical section on P-Q side {(qP'-qQ') / Fz * (Fz-d1) + qQ'}	X10	=	10.871 T/m²
Average pressure at critical section = {(X9+X10)/2}	Avpx3	=	10.262 T/m²
Average pressure at edge of the footing = {(qS'+qP')/2}	Avfx3	=	10.437 T/m²
Max. Shear force at Critical sec.={(Avpx3+Avfx3)*d1/2-d1*Tf*density*LF}	Sfz	=	12.400 T/m

Parallel to X-direction :

Distance of critical section from edge of footing along X-direction {dx-d}	d2	=	0.243 m
Pressure ordinate corr. to critical section on P-S side {(qP'-qS') / Fx * (Fx-d2) + qS'}	X11	=	10.891 T/m²
Pressure ordinate corr. to critical section on Q-R side {(qQ'-qR') / Fx * (Fx-d2) + qR'}	X12	=	10.376 T/m²
Average pressure at critical section = {(X11+X12)/2}	Avpz3	=	10.633 T/m²
Average pressure at edge of the footing = {(qP'+qQ')/2}	Axfz3	=	10.789 T/m²
Max. Shear force at Critical sec.={(Avpz3+Avfz3)*d2/2-d2*Tf*density*LF}	Sfx	=	2.056 T/m

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	Parallel to Z-direction	Parallel to X-direction
Shear Force	12.400 T/m	2.056 T/m
τv = Nominal Shear Stress	0.234 N/mm²	0.040 N/mm²
{Shear Force /(1000*d)}		
τc,max = Maximum shear stress	3.500 N/mm²	3.500 N/mm²
(Table 14,IS:456-1978)	<tcmax.SAFE IN SHEAR	<tcmax...SAFE IN SHEAR.
ß = {(0.8*fck)/(6.89*pt)} (cl. From SP-16)	23.977	23.423
τc = Design shear strength of concrete	0.289 N/mm²	0.292 N/mm²
[0.85*Sqrt.(0.8*fck)*(Sqrt.(1+5*ß)-1)] / [6*ß]	SAFE IN SHEAR.	SAFE IN SHEAR.

6.1.5.6.2 **Two way (Punching) Shear**

(Critical section is at a distance 'd/2'allround from the face of column)

Effective area {Af - (D+d)*(b+d)} = 7.727 m²

Punching force={(qP'+qS'+qQ'+qR')*Effec. area/4-Tf*Effec. area*Density*LF} = 61.277 T

τvp = Punching Shear Stress {Shear force / (2 * (D+d+b+d) * d)} τv = 0.327 N/mm²

ßc = Ratio of short side to long side of column = { b / D } ßc = 1.000

Ks={0.5 + ßc} [Max. 1.0] (Cl. 30.6.3.1 of IS:456-1978) Ks = 1.000

τc={0.25 * Sqrt(fck)} τc = 1.369 N/mm²

Permissible shear stress {Ks * τc} = 1.369 N/mm²

SAFE IN TWO WAY SHEAR.

6.2 STABILITY CHECKS FOR FOOTING

Mo ---> Overturning Moment

Mr ---> Restoring Moment = {0.9*(Load on column +DL of foundn.+Soil)*Lever arm}

FSo ---> Factor of safety against Overturning.

Load case	I	II	III
Mo-z axis (T-m)	1.100	11.600	1.100
Mr-z axis (T-m)	90.586	108.541	95.973
FSo-z	82.351	9.357	87.248
SAFE / UNSAFE	SAFE	SAFE	SAFE
Mo-x axis (T-m)	1.100	1.100	9.500
Mr-x axis (T-m)	214.546	257.071	227.304
FSo-x	195.042	233.701	23.927
SAFE / UNSAFE	SAFE	SAFE	SAFE

SUMMARY OF FOOTING DESIGN

Length of the footing along X-axis = 1900 mm

Length of the footing along Z-axis = 4500 mm

Thickness of footing = 600 mm

BOTTOM REINFORCEMENT

ALONG FZ ===> 12 @ 150 mm C/C

ALONG FX ===> 12 @ 150 mm C/C

TOP REINFORCEMENT

ALONG FZ ===> 10 @ 210 mm C/C

ALONG FX ===> 10 @ 210 mm C/C

Lateral reinforcement in footing not required

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6.3 **TABLE OF AXIAL LOADS & MOMENTS IN ASCENDING VALUE OF xu, FOR Mz**

ku	xu (mm)	e (m)	Pu/fckbD	Muz/fckbD²
0.1	40	0.079	-0.148	0.029
0.2	80	0.808	-0.035	0.071
0.3	120	0.841	0.044	0.092
0.456	158.232	0.841	0.044	0.092
0.4	160	0.382	0.107	0.102
0.5	200	0.237	0.177	0.105
0.6	240	0.162	0.249	0.101
0.7	280	0.111	0.323	0.090
0.8	320	0.081	0.387	0.079
0.9	360	0.059	0.444	0.066
1	400	0.041	0.495	0.051
1.1	440	0.030	0.528	0.040
1.2	480	0.023	0.549	0.032
1.28	510.61	0.020	0.560	0.028
1.276533302	510.6133207	0.023	0.549	0.032
1.33	532	0.018	0.567	0.025
1.5	600	0.013	0.582	0.019
1.75	700	0.009	0.596	0.014
2	800	0.003	0.622	0.004
1E+102	4E+104	0.000	0.627	0.000

6.4 **TABLE OF AXIAL LOADS & MOMENTS IN ASCENDING VALUE OF xu, FOR Mx**

ku	xu (mm)	e (m)	Pu/fckbD	Mux/fckb²D
0.1	40	0.079	-0.148	0.029
0.2	80	0.808	-0.035	0.071
0.3	120	0.841	0.044	0.092
0.456	158.232	0.287	0.146	0.105
0.4	160	0.382	0.107	0.102
0.5	200	0.237	0.177	0.105
0.6	240	0.162	0.249	0.101
0.7	280	0.111	0.323	0.090
0.8	320	0.081	0.387	0.079
0.9	360	0.059	0.444	0.066
1	400	0.041	0.495	0.051
1.100	440.000	0.030	0.528	0.040
1.2	480	0.023	0.549	0.032
1.277	510.613	0.020	0.560	0.028
1.276533302	510.6133207	0.023	0.549	0.032
1.33	532	0.018	0.567	0.025
1.5	600	0.013	0.582	0.019
1.75	700	0.009	0.596	0.014
2	800	0.007	0.604	0.011
2.5E+101	1E+104	0.000	0.627	0.000

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6.5 **SAMPLE CALCULATION FOR AXIAL LOAD AND MOMENTS FOR NEUTRAL AXIS POSITION i.e. xu = 1*D FROM THE MOST COMPRESSED FIBRE FOR BENDING ABOUT Z-AXIS. THE TABLE BELOW SHOWS THE CALCULATIONS FOR THE STRAINS AND STRESSES IN CONCRETE AND STEEL IN AT DIFFERENT ROW LEVELS.**

ku= 1.0 (i.e. Neutral axis inside the section)

xu=ku*D = 400 mm.

Puc=0.36 * fck * b * xu = 1728 KN

Muc=Puc*D*(0.5-0.416ku) = 58.061 KN-M

Counter for finding no. of rows = 4

Dist. between 2 bars = 98 mm.

Strain at different levels is calculated by interpolating between 0.0035 at the highly compressed edge to zero at N.A. Stresses in steel are interpolated for the corresponding strains and grade of steel from table A, Pg. 6 of SP-16. Stresses in concrete are calculated from the formula

fci=0.446 fck [2*ei/0.002 - (ei/0.002)²] for ei < 0.002 & fci=0.446 fck for ei >=0.002

Sr.No.	Steel area at diff. Row Asi (mm²)	Strains at different levels ei (N/mm²)	Stresses at diff. rows in steel fsi(N/mm²)	Stresses at diff. rows in concrete fci(N/mm²)
1	804.248	e1= 0.003	421.292 =fs1	fc1 = 13.380
2	402.124	e2= 0.002	385.613 =fs2	fc2 = 13.380
3	402.124	e3= 0.001	264.250 =fs3	fc3 = 11.839
4	804.248	e4= 5E-04	92.750 =fs4	fc4 = 5.486
5	0.000	e5= 0	0.000 =fs5	fc5 = 0.000
6	0.000	e6= 0	0.000 =fs6	fc6 = 0.000
7	0.000	e7= 0	0.000 =fs7	fc7 = 0.000
8	0.000	e8= 0	0.000 =fs8	fc8 = 0.000
9	0.000	e9= 0	0.000 =fs9	fc9 = 0.000
10	0.000	e10= 0	0.000 =fs10	fc10 = 0.000
11	0.000	e11= 0	0.000 =fs11	fc11 = 0.000

Sr.No.	Pusi =Asi*(fsi-fci)(KN)	yi (Dist. Of bars from CG) (mm)	Musi=(Pusi * yi) (KN-m)
1	328.062	147.000	48.23
2	149.684	49.000	7.33
3	101.501	-49.000	-4.97
4	70.182	-147.000	-10.32
5	0.000	0.000	0.00
6	0.000	0.000	0.00
7	0.000	0.000	0.00
8	0.000	0.000	0.00
9	0.000	0.000	0.00
10	0.000	0.000	0.00
11	0.000	0.000	0.00
SUM	649.428		40.27

Pu= Puc+Σ (Pusi) Mu=Muc+Σ (Musi)

242.347 T 10.023 T-m

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6.6 **SAMPLE CALC. FOR AXIAL LOAD & MOMENTS FOR NA POS. xu = 1.5*D FROM THE MOST COMPRESSED FIBRE FOR BENDING ABOUT Z-AXIS. TABLE BELOW SHOWS THE CALC. FOR THE STRAINS AND STRESSES IN CONC. AND STEEL IN AT DIFF. ROW LEVELS.**

ku=		1.5	(i.e. Neutral axis outside the section)
xu=ku*D	=	600.00	mm.
C1 = 0.446(1-(C3/6))	=	0.42	
Puc = C1 fck b D	=	2024.81	KN
C3=8/7(4/(7ku-3))^2	=	0.33	
C2=(0.5-C3/7)/(1-C3/6)	= =	0.48	
Muc = Puc *D*(0.5 - C2)	= =	16.57	KN-M
Counter for finding no. of rows	=	4	
Dist. between 2 bars	=	98.00	mm.

Strain at different levels is calculated by interpolating between 0.0035 at the highly compressed edge to zero at N.A. Stresses in steel are interpolated for the corresponding strains and grade of steel from table A, Pg. 6 of SP-16. Stresses in conc. are calculated using,

fci=0.446 fck [2*ei/0.002 - (ei/0.002)²] for ei < 0.002 & fci=0.446 fck for ei >=0.002

Sr.No.	Steel area at diff. Row Asi (mm²)	Strains at different levels ei (N/mm²)	Stresses at diff. rows in steel fsi(N/mm²)	Stresses at diff. rows in concrete fci(N/mm²)
1	804.248	e1= 0.003	403.753 =fs1	fc1 = 13.380
2	402.124	e2= 0.002	379.773 =fs2	fc2 = 13.380
3	402.124	e3= 0.002	327.600 =fs3	fc3 = 12.942
4	804.248	e4= 0.001	236.133 =fs4	fc4 = 11.134
5	0.000	e5= 0	0.000 =fs5	fc5 = 0.000
6	0.000	e6= 0	0.000 =fs6	fc6 = 0.000
7	0.000	e7= 0	0.000 =fs7	fc7 = 0.000
8	0.000	e8= 0	0.000 =fs8	fc8 = 0.000
9	0.000	e9= 0	0.000 =fs9	fc9 = 0.000
10	0.000	e10= 0	0.000 =fs10	fc10 = 0.000
11	0.000	e11= 0	0.000 =fs11	fc11 = 0.000

Sr.No.	Pusi =Asi*(fsi-fci)(KN)	yi (Dist. Of bars from CG) (mm)	Musi=(Pusi * yi) (KN-m)
1	313.956	147.000	46.15
2	147.336	49.000	7.22
3	126.532	-49.000	-6.20
4	180.955	-147.000	-26.60
5	0.000	0.000	0.00
6	0.000	0.000	0.00
7	0.000	0.000	0.00
8	0.000	0.000	0.00
9	0.000	0.000	0.00
10	0.000	0.000	0.00
11	0.000	0.000	0.00
SUM	768.778		20.57

Pu= Puc+Σ (Pusi) 284.770 T

Mu=Muc+Σ (Musi) 3.786 T-m