Maulana Azad National Institute of Technology, Bhopal

(An Institute of National Importance)

DEPARTMENT OF CIVIL ENGINEERING

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Ref. CE/ 2018 / ND/27 Date: 03-08-2018

MANIT PAN No. AAA TD5152 E

CGM-I
4.8.18

To
The Project Director
M. P. JAL NIGAM MARYADIT
Vindhyachal Bhawan
Bhopal, M.P.

Subject:- Checking of the Structural Design & Drawings of the RCC ESR for various locations.

Sir,

With reference to the above, regarding the Checking of the Drawings of the RCC ESR for the standard conditions of :

- Seismic Zone location = 3; Soil strata having s.b.c. of 7.50 T/m^2, 10.0 T/m^2, 15.0 T/m^2 & 20.0 T/m^2; Staging Height = 12 meters; Tank Capacity = 100 Kl
- Seismic Zone location = 3; Soil strata having s.b.c. of 7.50 T/m^2, 10.0 T/m^2, 15.0 T/m^2 & 20.0 T/m^2; Staging Height = 12 meters; Tank Capacity = 125 Kl
- Seismic Zone location = 3; Soil strata having s.b.c. of 7.50 T/m^2, 10.0 T/m^2, 15.0 T/m^2 & 20.0 T/m^2; Staging Height = 12 meters; Tank Capacity = 150 Kl
- Seismic Zone location = 3; Soil strata having s.b.c. of 7.50 T/m^2, 10.0 T/m^2, 15.0 T/m^2 & 20.0 T/m^2; Staging Height = 12 meters; Tank Capacity = 175 Kl

The design calculations and the drawings are found to be as per the prevailing code of practice.

This letter is issued on the clear understanding that our responsibility of the vetting of the designs of the foundation of above mentioned project and its proper structural performance will cease the moment any additions or alterations to the structural member / foundation by accident or due to tempering by the users for any reasons whatsoever, change of use, changes in location & size of walls is done.

Our responsibility will also cease in the event of non standard material, bad workmanship, overloading or lack of proper maintenance of structure / foundation or any such act, which is detrimental to the structure.

Thanks

3/8/18

Nitin Dindorkar
Professor, Civil Engg. Deptt.

PROFESSOR
Department of Civil Engineering
Maulana Azad National Institute of Technology
BHOPAL-462007 (M.P.)

1.75 LAKH LIT CAPACITY RCC ESR OF 12 M STG HT FOR MADHYA PRADESH REGION

Client : PROJECT DIRECTOR
M.P. JAL NIGAM MARYADIT

Agency :

Struct Design by : **M/s Aquades Structural Consultants (P) Ltd**
Hariram Residency Wing B
B- 403, Plot No 402,
Lft Khonde Marg, Khare Town,
Dharampeth NAGPUR - 440 010
☎ (O) 0712 2542726 (F) 2553577
(R) 0712 2245150, 2225485

email : aquades@rediffmail.com

Data :

Capacity	:	175 m^3
FSL	:	15.7 m
LDL	:	12.0
GL	:	0.00
FL	:	- 3.0
Free Board	:	0.5 m
SBC of soil	:	10/ 15/ 20 t/m^2 & 7.5 t/m^2
Seismic Zone	:	III

Reference :

1) IS – 456/2000
2) IS - 875/2015
3) IS - 1893/2002
4) IS - 3370/2009, Part I to II, IS 3370/1967- Part III & IV
5) IS - 13920/1993
6) Plain and Reinforced concrete, Vol, I & II, Jain and Jaikrishna
7) R C Designer's Handbook by Reynolds

Ref Drg : ASCPL/18023/17, 18 dt - 31/5/2018

Permissible Stresses: As per IS 3370: 2009, IS 456 : 2000

ESR ON 5 COLUMNS **1.75 LAKH LTRS ESR 12 M staging**

CAPACITY	175		
FSL	15.7 M		
LDL	12 M		
GL	0 M	STG HT	12 M
FL	-3 M		
FREE BOARD	0.5 M		
SBC	15 T/M^2		
SEISMIC ZONE	III		

CAPACITY CALCULATIONS

Water depth= 15.7-12 = 3.7 m

Consider the container to supproted on 4 nos of interior columns and 8 nos of priphery columns

consider column inside container 250 X 250

Area= 175/3.7+ 1 X 0.0625 47.359797 m^2

clear radius = 3.88266554 say = **3.885**

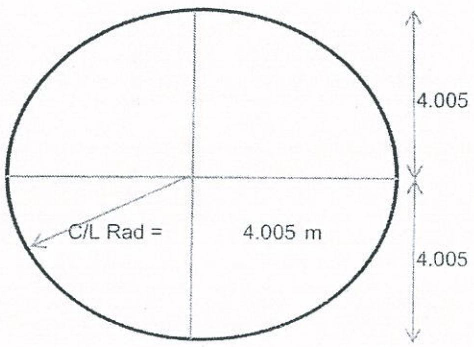
Assume wall thickness = 200 mm

Hence centre line radius = **4.005** Clear R= 3885 mm ie 3.885

Capacity of cont= 175.211 Cum OK

DESIGN OF ROOF SLAB **120** THK M300 CONC

Span= 4.005 X 4.005



DL=	300 kg/m^2
Water proofing=	100 kg/m^2
Live load=	75 kg/m^2
Total=	475 kg/m^2

Effective span = $\text{sqrt}\left(\dfrac{\pi \; (4.005)^2}{4}\right) =$ 3.55

Refer Table 22 IS 456/2000, Case 4 - two adjacent edges discontinuous

$\dfrac{Ly}{Lx} = 1$

-ve BM = 0.047 X 475 X 3.55^2 = 282 Kgm

+ve BM = 0.035 X 475 X 3.55^2 = 210 Kgm

Wt of central shaft = 500 Kg assumed

BM due to Cntral Shaft = 30 Kgm say

Net BM = 282 + 30 = 312 Kgm

Verified
3/8/18

PROFESSOR

$$As = \frac{31200}{0.87 \times 1500 \times 7.5} = 2.52 \text{ cm}^2$$

Mini As = 0.313 X 12 = 3.756 cm^2

Provide **8 tor @** **130 c/c** bothways in square mesh at bottom, alt the bar bent up at l/5 800 from support 3.869 cm^2 OK

Provide extra at top **8 tor @** **260 c/c** upto l/4 1000 from support.

CHECK FOR VERTICAL DEFLECTION THICKNESS 120

As provided 8 130 c/c 3.869 cm^2

$$\frac{100\,As}{b\,d} = \frac{3.87}{7.5} = 0.52$$

From fig (3), IS 456/2000, Modification factor = 1.5

Clause 22.2.1, span / depth = 26 X 1.5 = 39

$$\text{Hence depth required} = \frac{3550 - 250}{39} = 84.61538 \text{ OK}$$

Check for CRACK WIDTH

Refer Appendix CC1, WC = 0.0621703 << 0.2 mm OK

APPENDIX CC1 BM IN ROOF SLAB

CRACK WIDTH CALCULATIONS

DATA

Permissible crackwidth		0.2 mm	Ms =	3.12 kNm			
Width of the section	b	1000 mm	Mu=	4.68 kNm			
Overall Depth of the section	D	120 mm	Cover	45 mm			
Es		2.00E+05 Mpa					
fck		30 Mpa	Ec = 5000*sqrt(30)=	27386.13 Mpa	As req=	189.6663	
Use	8 dia bars @ 130 mm c/c		*(For calculation of crackwidth use 0.5 Ec)*				
As =	386.66 sqmm per 1000mm	Fy = 415	Mpa			total steel	386.923 sqmm
a_{cr} =	77.40					Spacing	130 c/c
d=(D-c-dia/2) =	71.00 mm	(CHECK)				Eq Dia	8.0 cm

r = 0.005446 , Modulus of elasticity m = 14.60593

(x/d) = -mr+sqrt(mr(2+mr)) = 0.327166 ie x= 23.23 mm

For design crackwidth = 0.2 mm

e2 = (b (D-x) (a'-x)/(3 Es As (d-x), where for crack width at tension suface a'=D

As such e2 = 0.00084498

For design crackwidth = 0.1 mm

e2 = (1 5 x b (D-x) (a'-x)/(3 Es As (d-x), where for crack width at tension suface a'=D

As such e2 = 0.00126748

For our case e2 = 0.0008

A= (Ms/(Es*As*(d-(x/3)))*(h-x)/(d-x) = 1.29E-03

B= (1+2*(acr-c)/(h-x)) = 1.669626

W=(A-e2)*3*acr/(B) = 0.0621703

LA Z = d - (x/3) = 63.26

Steel Tensile Stress fs = Ms/(z As) = 127.561 M Pa

Conc Comp Stress = 2Ms/(z b x) = 4.247 M Pa

CHECK STRESS LEVELS

0.45 fcu = 13.5 M Pa OK

0.8 fy = 332 M Pa OK

ROOF BEAM 250 X 300 span= 4.005 m M300 Conc

$$\text{Load on one beam} = \frac{475 \times 4.105^2 \times p}{8} = 3144 \text{ Kg}$$

Triangular

udl = 0.25 X 0.18 X 2500 = 112.5 Kg/m

$$SF = \frac{3144 + 112.5 \times 4.005}{2} = 1798 \text{ Kg}$$

$$-veBM = \frac{5}{48} \times 3144 \times 4.005 + \frac{112.5 \times 4.005^2}{12} - \frac{1798 \times 0.25}{3}$$

= 1312 + 151 - 150

= 1314 Kgm

+veBM = 0.6 X 1312 + 0.5 X 151 = 863 Kgm

$$-veAs = \frac{131400}{0.87 \times 1900 \times 25} = 3.18 \text{ cm}^2$$

$$+veAs = \frac{86300}{0.87 \times 1500 \times 25} = 2.645 \text{ cm}^2$$

$$As\ min = \frac{0.85}{415} \times 25 \times 25 = 1.281 \text{ cm}^2$$

Provide at bottom 2 X 12 tor str
1 X 12 tor bent up @ 800

Provide at top 2 X 12 tor str

$$\tau_v = \frac{1798}{25 \times 25} = 2.88 \quad \text{Kg/cm}^2$$

$$\frac{100\,As}{b\,d} = \frac{3.3929 \times 100}{25 \times 25} = 0.543$$ Hence tau c = 3.203 Kg/cm^2

Net SF = 1798 - 3.203 X 25 X 25 = -204 kg

Max Space= 18.75

Provide 8 tor @ stps @ 175 c/c

COLUMN INSIDE CONTAINER 250 X 250 M300 Conc

Load from Beam = 1798 X 4 = 7192 Kg

self wt = 0.0625 X 4.2 X 2500 = 657 Kg

7849 Kg

L= 4.2 , d = 0.25

L/d = 10.92 SHORT COLUMN OK

R =1.25 - (L/d)/48 = 1.022 R= 1

Design Load =7849/ 1 = R= 7849 Kg Very Small

Provide 4 X 12 tor Main Bars

Provide 8 tor links @ 150 c/c

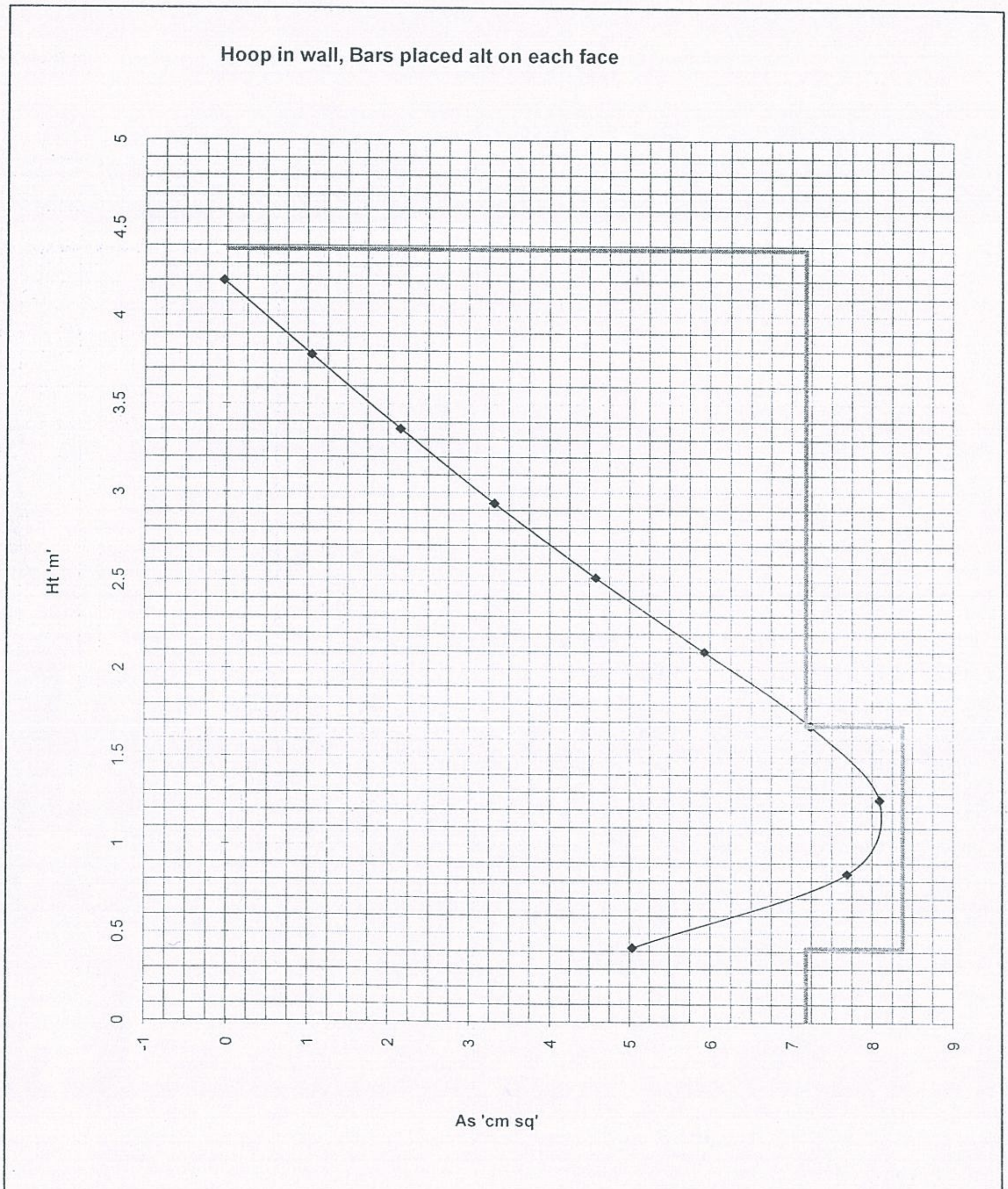
H=	4.2	D=	7.77	t=	0.2	(m-1)=	8
w =	1000						
H^2/Dt=	11.35135	fact	1.351351	fact2	2		

175 Cum capacity RCC ESR water depth 3.7 m free board 0.5m

APPENDIX - Ia

Ref : Table 12 of IS 3370(IV), for hoop tension in cylindrical wall

	0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1
factor	-0.0039	0.0964	0.1980	0.3049	0.4206	0.5446	0.6646	0.7435	0.7064	0.4627	
T , kg=	-64	1572	3230	4975	6862	8886	10845	12132	11526	7550	0
As =	-0.043	1.048	2.154	3.317	4.575	5.924	7.230	8.088	7.684	5.034	
Ht=	**4.2**	**3.78**	**3.36**	**2.94**	**2.52**	**2.1**	**1.68**	**1.26**	**0.84**	**0.42**	
Hta	-0.0322	0.782801	1.601368	2.455108	3.369539	4.340017	5.270123	5.875861	5.591139	3.70067	



DESIGN OF WALL 200 THICK

Analysis of bottom joint of wall,base slab,b r beam & gallery

Wall :
****

*C LRadius 'R' m :	4.005
*Thkof wall 'Tw' m :	0.2
*Htof Water 'h' m :	4.2
Hoop Ten 'HT' Kgm :	16317
Disp"HT*R/TwE" 'd' :	326747.925
Slope"d/h" 'Theta' :	77797.125
u "(3/(RTw)^2)^.25":	1.470496705
Z = "(ETw^3/12)" :	0.000666667
Mom Stiffness"2uZ" :	0.001960662
Cor Thrust"2 u^2 Z":	0.002883147
Thu Stiff "4 u^3 Z":	0.008479318

Base Slab :
****

*Thk of slab 'Ts' m:	0.25
Eff Width"6Ts" 'Be':	1.5
Thu Stif "EA/R1R2":	0.029677641
M St"EATs^2/12R1R2":	0.000154571

Bottom Ring Beam :
****

Size

* 'b' m:	0.3
* 'd' m:	0.7
Thu Stif EA/R^2 :	0.013092249
M St "EAd^2/12RR" :	0.0005346

Gallery :
****

*Width 'w' m :	0
*Thickness 'Tg' m :	0
Thu Stif "EA/R1R2":	0
MSt"EA*Tw^2/12R1R2":	0

Continuity Analysis :
****

Member	Slope		Displ		Moment	Thrust
Wall	0'-	77797.1	y'-	326747.9	MSt*Sl-CorThu	CorThu*Sl+ThSt*Disp
Base Slab	0'		y'		MSt * Sl	ThuSt*Disp
Bot Ring Beam	0'		y'		MSt * Sl	ThuSt*Disp
Gallery	0'		y'		MSt * Sl	ThuSt*Disp

Sumation of moment = 0 & Thrust = 0 implies

	0'	y'	= Const
Eq1=> Sum of Mom=0:	0.002649833	-0.0029	-789.5285
Eq2=> Sum of Thu=0:	-0.002883147	0.05125	2546.2988

From Eqation 1 & 2

Invert of above	401.9882182	22.6148
matrix	22.61481373	20.7847

0'=	-259797.1
y'=	35069.137

HT in wall =	1751.267767
HT in B Slab =	4168.280947
HT in BotRingBeam =	1838.831155
HT in Gallery =	0
BM in wall =	179.0446825
BM in B Slab =	-40.1571103
BM in BotRingBeam =	-138.8875722
BM in Gallery =	0

DESIGN OF WALL **200 THICK** M300 Conc

Ht at Base = 1751.2678 very small

$$6o = \frac{1751.2678}{2000} = 0.88\ \text{Kg/cm}^2 \quad < 13\ \text{OK}$$

Provide steel as below- (Bars to be placed alternately on each face

Base to	420	8 tor	@ 70 c/c	7 nos	7.181	cm2
420 to	1680	8 tor	@ 60 c/c	21 nos	8.378	cm2
1680 to	4380	8 tor	@ 70 c/c	38 nos	7.181	cm3

VERTICAL STEEL-
Refre IS 3370 (IV) Table 13
coefficient = 0.004094595

BM = 0.004094595 X 4200 X 4.200^2 = 303.37 Kgm cl

cl

$$6bt = \frac{303.37 \times 6}{400} = 4.56\ \text{Kg/cm}^2$$

< 13 Hence OK

$$As = \frac{303.37 \times 100}{.87 \times 1500 \times 14.5} = 1.61\ \text{cm}^2$$

Mini As = 4.8 cm^2
Provide on each face 8 tor @ 150 c/c

Area
335.11

Ref Apendix CC2 for crackwidth check for Hoop in wall. Wc = **-0.026634 << 0.2 mm OK**

Ref Apendix CC3 for crackwidth check for Vert BM in wall. Wc = **-0.122877 << 0.2 mm OK**

DESIGN FOR DIRECT TENSION		APPENDIX CC2	HOOP IN WALL	
Tension =	121319 N (Working)			
Concrete M	30 M Pa	Es	2.00E+05 M Pa	
Steel	415 M Pa	C	0.0003	
Width	1000 mm	Ec	27386.13 M Pa	
Depth	200 mm	m = (Es/(Ec*0.5) =		14.606
Clear Cover	40 mm	m=(280/3 6cbc) =		9.333
Req As = 1.5* HT /(0.87 * fy*.9) =	560.03	Lim W =	0.2	
Provided Steel				
8 dia bars @	60 mm c/c	Cover=	44 mm (CHECK)	
	837.75804 sqmm	ok		

Check Tensile Stress in Concrete

fct = (C Es Ast + HT)/(Ac+(m-1) Ast) = 0.83 M pa Less Than 3 M Pa, OK

Calculation of Crack width

Apparent Strain e1 = T / As Es

e1 = 0.00072407

For design crackwidth = 0.2 mm

e2 = 2 b d/ (3 Es As)

As such e2 = 0.000795775

For design crackwidth = 0.1 mm

e2 = b d / (Es As)

As such e2 = 0.0011937 sqmm

For our case e2 = 0.0008

Avg Strain em = e1 - e2

e m = -7.17047E-05

a cr = 123.81

Crackwidth = 3 a cr em = -0.026634

ok

APPENDIX CC3 BM IN WALL

CRACK WIDTH CALCULATIONS

DATA

Permissible crackwidth			0.2 mm	Ms =	3.034 kNm			
Width of the section		b	1000 mm	Mu=	4.551 kNm			
Overall Depth of the section		D	200 mm	Cover	45 mm			
Es			2.00E+05 Mpa					
fck			30 Mpa	Ec = 5000*sqrt(30)= 27386.13 Mpa		As req=	84.16689	
Use	8 dia bars @ 150 mm c/c			*(For calculation of crackwidth use 0.5 Ec)*				
As =	335.10 sqmm per 1000mm	Fy =	415	Mpa		total steel	335.11 sqcm	
a_{cr} =	85.59					Spacing	150 c/c	
d=(D-c-dia/2) =	151.00 mm	(CHECK)				Eq Dia	8.000081 cm	
r =	0.002219227 , Modulus of elasticity **m** =		14.60593					
(x/d) = -mr+sqrt(mr(2+mr)) =	0.224254 ie x=		33.86 mm					

For design crackwidth = 0.2 mm

e2 = (b (D-x) (a'-x)/(3 Es As (d-x), where for crack width at tension suface a'=D

As such e2 = 0.001171952

For design crackwidth = 0.1 mm

e2 = (1.5 x b (D-x) (a'-x)/(3 Es As (d-x), where for crack width at tension suface a'=D

As such e2 = 0.001757928

For our case e2 = 0.0012

A= (Ms/(Es*As*(d-(x/3)))*(h-x)/(d-x) =	4.60E-04
B= (1+2*(acr-c)/(h-x)) =	1.48860626
W=(A-e2)*3*acr/(B) =	-0.12288

LA Z = d - (x/3) =	139.71		
Steel Tensile Stress fs = Ms/(z As) =	64.804	M Pa	
Conc Comp Stress = 2Ms/(z b x) =	1.283	M Pa	
CHECK STRESS LEVELS			
0.45 fcu =	13.5 M Pa	OK	
0.8 fy =	332 M Pa	OK	

BASE SLAB 250 THICK M300 CONC

SPAN = **4.005 X 4.005**

Loading =

DL	=	625
water	=	4200
plaster	=	42

		4867 Kg/m^2

Refer table 22, IS 456/2000, Interior Panel

$\dfrac{Ly}{Lx}$ = 1 ,Equivalent Panel = 3.55

- ve BM = 0.032 X 4867 X 3.550^2 = 1963 Kgm

+ve BM = 0.024 X 4867 X 3.550^2 = 1473 Kgm

Total -ve BM = 1963 + 139 = 2102 Kgm

Total +ve BM = 1473 + 139 = 1612 Kgm

-ve As = $\dfrac{210200}{0.87 \times 1500 \times 20}$ = 8.054 cm^2

+ve As = $\dfrac{161200}{0.87 \times 1500 \times 20}$ = 6.177 cm^2

or Mini As = 0.313 X 25 = 7.825 cm^2

provide at bottom **10 tor @** **95 c/c** bothways in square mesh at bottom, alternate
bar bent up @ l/5 **800** from support 8.2673 c/c cm^2

provide at top **10 tor @** **190 c/c** BOTHWAYS l/4 4.1337 c/c

-ve As provided = 8.268 cm^2 OK

+ve As provided = 8.267 c/c cm^2 OK

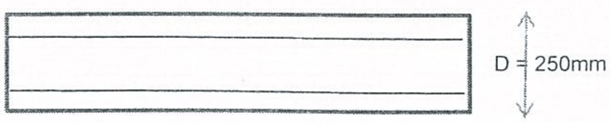
Hoop in base slab = $\dfrac{4169}{1500}$ = 2.779 cm^2

provide 2 X **10 tor** hoop bars at top & bottom near wall (As provided=3.141cm^2) Total 4

CHECK FOR NON CRACKING

Ast=66.144

Asb=51.68



Area of steel provided at top= (9 - 1) X 8.268 = 66.144 cm^2

Area of steel provided at bottom= (1.5 X 9 -1) X 4.1336745 51.68 cm^2

$$A_s = (25) \times 100 + 66.144 + 51.68 = 2617.824 \text{ cm}^2$$

$$y = \frac{2500 \times 12.5 + 66.144 \times 5.5 + 51.68 \times 19.5}{2617.824} = 12.47 \text{ cm}$$

$$I = \frac{100 \times 25^3}{12} + (2500)*(12.5 - 12.47)^2 + 66.144 \times 6.97^2 + 51.68 \times 7.03^2$$

$$= 130208.3 + 2.25 + 3213.335 + 2554.072$$

$$= 135978 \text{ cm}^4$$

$$Z = 10904.4098 \text{ cm}^3$$

$$6bt = \frac{210200}{10904.4098} = 19.28 \text{ Kg/cm}^2 \quad < 20 \text{ hence OK}$$

CHECK FOR CRACK WIDTH

Refer crackwidth check vide App CC4

Wc= 0.0689488 < 0.2 mm OK

APPENDIX CC4 BM IN BASE SLAB

CRACK WIDTH CALCULATIONS

DATA

Permissible crackwidth		0.2 mm	Ms =	21.02 kNm			
Width of the section	b	1000 mm	Mu=	31.53 kNm			
Overall Depth of the section	D	250 mm	Cover	45 mm			
Es		2.00E+05 Mpa					
fck		30 Mpa	Ec = 5000*sqrt(30)=	27386.13 Mpa	As req=	450.9266	
Use	10.001 dia bars @ 95 mm c/c		*(For calculation of crackwidth use 0.5 Ec)*				
As =	826.90 sqmm per 1000mm	Fy = 415	Mpa			total steel	826.8 sqcm
a_{cr} =	63.97					Spacing	95 c/c
d=(D-c-	200.00 mm	(CHECK)				Eq Dia	10.001 cm

r = 0.004134512 , Modulus of elasticity m = 14 60593

(x/d) = -mr+sqrt(mr(2+mr)) = 0.2923489 ie x= 58.47 mm

For design crackwidth = 0.2 mm

e2 = (b (D-x) (a'-x)/(3 Es As (d-x), where for crack width at tension suface a'=D

As such e2 = 0.00052242

For design crackwidth = 0.1 mm

e2 = (1.5 x b (D-x) (a'-x)/(3 Es As (d-x), where for crack width at tension suface a'=D

As such e2 = 0.00078364

For our case e2 = 0.0005

A= (Ms/(Es*As*(d-(x/3)))*(h-x)/(d-x) = 9.53E-04

B= (1+2*(acr-c)/(h-x)) = 1.198041

W=(A-e2)*3*acr/(B) = 0.0689488

LA Z = 180.51

Steel Tensile Stress fs = Ms/(z As) = 140.825 M Pa

Conc Comp Stress = 2Ms/(z b x) = 3.983 M Pa

CHECK STRESS LEVELS

0.45 fcu = 13.5 M Pa OK

0.8 fy = 332 M Pa OK

BASE BEAM 300 X 700 span= 4.005 m

M300 CONC

Load from slab = $\dfrac{4867 \times 4.005^2 \times p}{8}$ = 30657 Kg

udl = 0.3 X 0.45 X 2500 = 337.5 Kg/m

SF = $\dfrac{30657 + 337.5 \times 4.005}{2}$ 16005 Kg

-veBM = $\dfrac{5}{48} \times 30657 \times 4.005 + \dfrac{337.5 \times 4.005^2}{12} - \dfrac{16005 \times 0.5}{3}$

= 12790 + 452 - 2668

= 10575 Kgm

+veBM = 0.6 X 12790 + 0.5 X 452 = 7900 Kgm

-veAs = $\dfrac{1057500}{0.87 \times 1500 \times 65}$: 12.467 cm^2

+veAs = $\dfrac{790000}{0.87 \times 1900 \times 65}$ = 7.353 cm^2

As min = $\dfrac{0.85}{415} \times 30 \times 65$ = 3.994 cm^2

Provide at bottom 2 X 16 tor str
2 X 20 tor extra in centre piece of 2805 10.30
OK

Provide at top 2 X 16 tor str
3 X 20 tor extra near supp upto 1000 13.45
OK

tau v = $\dfrac{16005}{30 \times 65}$ = 8.21 Kg/cm^2

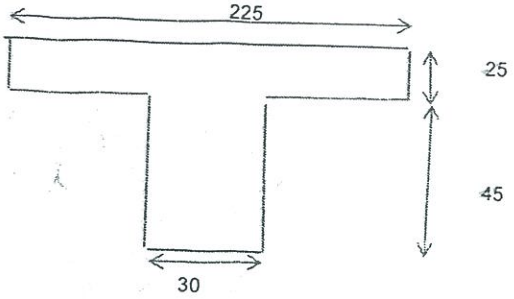
$\dfrac{100\ As}{b\ d}$ = $\dfrac{13.45 \times 100}{30 \times 65}$ = 0.690 Hence tau c = 3.56 Kg/cm^2

Net SF = 16005 - 3.56 X 30 X 65 = 9063 kg

8 tor stps @ $\dfrac{1.00531 \times 1750 \times 65}{9063}$ = 12.618 say 120

supp to 960	960	8 tor	stps @	120 c/c
960	to centre	8 tor	stps @	200 c/c

NON CRACKING CHECK



$$bf = \frac{Lo}{6} + bw + 6\ df$$

$$= \frac{0.7 \times 4.005}{6} + 0.3 + 6 \times 0.25 = 2.268 \text{ m}$$

say 225 cm

$$A = 5625 + 1350 = 6975 \text{ cm}^2$$

$$Y = \frac{5625 \times 12.5 + 1350 \times 47.5}{6975} \quad 19.275 \text{ cm}$$

$$I = \frac{225}{12} \times 25\text{^}3 + 5625 \times 6.775\text{^}2 + \frac{30}{12} \times 45\text{^}3 + 1350 \times 28.225\text{^}2$$

$$= 292969 + 258191.02 + 227812.5 + 1075478 = 1854450.61 \text{ cm}^4$$

$$Z = \frac{1854450.609}{19.275} = 96211 \text{ cm}^3$$

$$6bt = \frac{1057500}{96211} \quad 10.992 \textbf{ OK}$$

CIRCULAR RING BEAM BELOW WALL 300 X 700 M300 R= 4.005 m

span= 6292 mm

BM in BRB = 139 Kgm

Radial BM = 139 X 4.005 = 557 Kgm

This Beam will take load when wall is not constructed.

Roof Slab =	(625 + 300)3.14X4.105)^2/8 =	6122	Kg
Self =	2X3.14X4.005/4 X 0.3 X 0.45 X 2500 =	2124	Kg
Gallery =	3.14 X (5.105^2 -4.105^2)600/4 =	4341	Kg
		12587	

It is supported on 4 columns, $2\alpha = \frac{\pi}{4}$

$$WR = \frac{12587}{3.14/2} = 8014$$

$WR^2(2\alpha)$ = 8014 X 4.005 X 3.14 /2 = 50417 Kg

-ve BM = 0.137 X 50417 = 6908 Kgm

+ve BM = 0.07 X 50417 = 3530 Kgm

T = 0.021 X 50417 = 1059

SF at support = 8014 X 3.14 /4 = 6295 Kg

SF at contra = WR(α - Θ)

$$= 8014 \times \left(\frac{\pi}{4} - 0.336\right) = 3602 \text{ Kg}$$

$$\text{-ve Net BM} = 6908 + 557 - \frac{6295 \times 0.4}{3} = 6626 \text{ Kgm}$$

+ve Net BM = 4087 Kgm

$$\text{- ve As} = \frac{662600}{0.87 \times 1900 \times 65} = 6.167 \text{ cm}^2$$

$$\text{+ ve As} = \frac{408700}{0.87 \times 1500 \times 65} = 4.819 \text{ cm}^2$$

$$\text{As min} = \frac{0.85}{415} \times 30 \times 65 = 3.994 \text{ cm}^2$$

Provide at bottom 3 X 12 tor str
2 X 12 tor extra in the centre piece of length 0.7 l 5.65 Sqcm
OK

Provide at top 3 X 12 tor str
0 X 0 tor + 2 X 16 tor extra near support upto L/4 7.42 Sqcm
OK

$$Ve = 3602 + \frac{1059 \times 1.6}{0.3} = 9250.00 \text{ Kg/cm}^2$$

$$\frac{100\,As}{b\,d} = \frac{7.415 \times 100}{30 \times 65} = 0.380$$ Hence tau c = 2.72 Kg/cm^2

Net SF = 6295 - 2.72 X 30 X 65 = 991 kg

$$8 \text{ tor stps @ } \frac{1.006 \times 1750 \times 65}{991} = 115.48 \text{ cm c/c say } 200 \text{ mm}$$

Provide 8 tor stps @ 200 c/c (these bars to be taken straight in Gallery)

DESIGN OF GALLERY 120 THICK Span = 1 m cantilever

DL = 300 M300 Conc

LL = 300

Total = 600 Kg/m^2

BM = 300 Kgm

$$As = \frac{30000}{0.87 \times 1900 \times 7.6} = 2.389\ cm^2$$

Provide at top 8 tor @ 200 c/c radial bars ***every alternate*** rod chim 2.514 OK

(there are stp bars from bottom ring beam)

Circumferential bars 4 X 8 tor at top and bottom

Check for wall as beam 200 wide X 5200 deep

It is tied at top by roof slab and at bottom by base slab

$$\text{Roof Slab} = \frac{PI\ x475\ \ X\ 4.105\ \ \hat{}2}{8} = 3143\ Kg$$

$$\text{Base Slab} = \frac{PI\ x4867\ \ X\ 4.105\ \ \hat{}2}{8} = 32207\ Kg$$

:lf Wt = (0.2x4.38+0.3 x0.45) x2500 x 2x pix4.005/4 15901 Kg

Gallery = 17360 Kg

TOTAL 68612 Kg

$$WR^2\ (2a) = \frac{68612\ X\ \ 4.005}{1} = 274792$$

-ve BM = 0.137 X 274792 = 37647 Kgm

+ve BM = 0.07 X 274792 = 19236 Kgm

Torsion = 0.021 X 274792 = 5771 Kgm

SF at support = 68612 X 1/2 = 34306 Kg

WR = 68612/ PI() *0.5 = 43680

$$\text{SF at contra} = 43680\ X\ \ 1\ X\ (\ \frac{3.142}{4} - 0.336) = 19629.71\ Kg$$

$$6bt = \frac{3764700\ \ X\ 6}{20\ X\ \ 520\hat{}2} = 4.177\ Kg/cm^2$$

< 20 hence OK

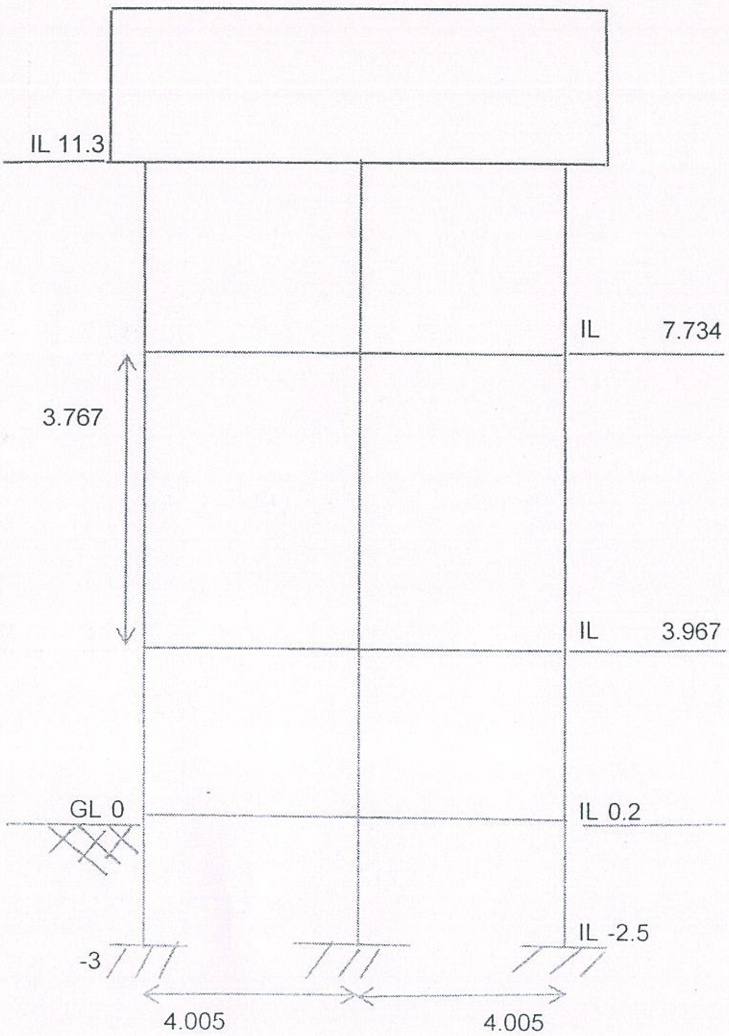
$$Ve = 19629.712 + \frac{5771\ X\ 1.6}{0.2} = 65798\ Kg$$

$$Tau\ v = \frac{65798}{20\ X\ 515} = 6.388127\ Kg/cm^2$$

very small

Hence O K

Staging -	consider	500	dia Interior columns	1 nos	brace	300	400
		500	dia Periphery columns	4 nos	no of br lvls=		3



INTERIOR COLUMN **500** DIA 1 Nos

$$A = \frac{3.1416 \times 50^2}{4} = 1963 \text{ cm}^2$$

$$I = \frac{3.1416 \times 50^4}{4} = 4908739 \text{ cm}^4$$

PERIPHERY COLUMI **500** DIA 4 Nos

$$A = \frac{3.1416 \times 50^2}{4} = 1963 \text{ cm}^2$$

$$I = \frac{3.1416 \times 50^4}{4} = 4908739 \text{ cm}^4$$

Brace **300 X 400** span= 4.005

A = 0.3 X 0.4 = 0.12 cm^2

Assume an arbitrary force of 10 tonnes acting at cg of container. Load shared by frame will be proportional to stiffness of its column to the total stiffness of structure.

Stiffness of Structure = 1 X 490.8739 + 4 X 490.8739 = 2454.37

Stiffness of one frame = 1 X 490.8739 + 2 X 490.8739 = 1472.622

Force on one frame = $\frac{1472.622}{2454.37}$ X 10000 = 6000 Kg

The frame is analysed on STRAPi and the member forces are shown in appendix I

E= 5000 X sqrt (25) 25000 N/mm^2

deflection= 0.01628 m

WEIGHT OF CONTAINER

R =	**4.005**			R slab = **0.12**		Wall = **0.2**	
Wall height =			**3.7** panel size	**4.005**	Cant span		**0**
R Beam		b= **0.25**		d = **0.3**		FB= **0.5**	
B Slab =	**0.25**	B Beam =		b = **0.3**		d = **0.7**	
Gallery =	d=	**0.12**		b =	**1** col pery =	**0.5**	
col int =	**0.5**		LL = **75**		Finish = **100**		
col inside container		**0.25**	**0.25**		Haunch=	**0**	**0**
circular beam below wall b=			**0.3**		d= **0.7**		
plaster= **42**							

Roof Slab	3.14xR^2xLoad on roof slab =					25146 Kg
R Beam =	0.25 X	0.18 X 2 X	8.01 X 2500			1802 Kg
Wall	2X3.14X4.005	X 4.38 X 2500	X 0.2			55110 Kg
col inside container	1 X 4.2 X	0.0625 X 2500				656 Kg
Base Slab	3.14xR^2xLoad on Base slab =					33087 Kg
B Beam	0.3 X	0.45 X 2 X	8.01 X 2500			5407 Kg
Circ beam	0.3 X	0.45 X 2 X	pi() x R			8493 Kg
Ring Beam	2X3.14X4.005	X 0.3 X	0.45 X 2500			8493 Kg
Gallery	3.14 (5.105^2	- 4.105 ^2) X	600			17360 Kg
Water	(3.14 x 3.885^2 X	4.2 - 1 X 3.3 X	0.0625)X1000			198888 Kg
Plaster	(2X3.14X3.895	X4.38+3.14X	3.895^2)x.02x	2080		6442 Kg
			Wt of container			**352391** Kg

WEIGHT OF STAGING

BRACE	**0.3**	**0.4**	stg	**12**	no of br lvl	**3**
Column	3.14(4 X .250^2	+ 1x 0.25^2)X	13.8 X2500			33870 Kg
Brace	0.12 (4 X (	3.505 +	5.164)) X 2500 X 3			31208 Kg
			wt of staging			**65079** Kg

ADD FOR STAIR

MB1 =	10 X	3076 =	30760		
Total Staging Weight =		65079 +	30760 =	**95839** Kg	

LL on Container

R Slab	3.14 X 4.105^2 X 75			3970 Kg
Gallery	3.14 (5.105^2	- 4.105 ^2) X	300	8680 Kg
			LL on container	**12651** Kg

Net Load on Container =		352391 - 12651 =	**339740** Kg
Deduct LL on stair =	300 / 980 x 30760 =		9416
Net Staging Load =	95839 - 9416 =		**86423** Kg

Wt of container on interior col

col inside cont			7849 Kg
Base Beam=	16005 x4		64020 Kg
		Non Seismic	**71869** Kg
Net Axial load	71869 - 4.005^2X75X3.14/5		**71113** Kg
			(Seismic)

Wt of staging on interior col

Col	3.14X 0.25^2 X 13.8 X	2500		6774.06 Kg
Brace	0.12 X 3.505 X 2 X	3 X 2500		6309.00 Kg
			Total	**13083** Kg

Wt of container on Periphery col

Axial load	1/4(352390.85 - 71869)	70130 Kg
Net axial load	1/4 X (339740 - 71113.132)	67157 Kg

Wt of staging on periphery col

	1/4 X (95839 - 13083.059)	20689 Kg
Net wt of staging on periphery column =	1/4 X (86423 - 13083)	18335 Kg

Confining Reinforcement

Interior col 500 dia

Ash= 0.09xSxDkx(fck/fy)x(Ag/Ak-1)
stp dia 8
S= spacing of links
Fck = 25 Mpa
Fy= 415
Ag = 500 ^2
Ak = 436 ^2
Dk= 436

S= 67.48 say= 66 mm

provide 8 tor stps @ **66** c/cnear joint for the height of **584**

Periphery column 500 dia

stp dia 8
Ag = 500 ^2
Ak = 436 ^2
Dk= 436
S= 67.48 say= 66
provide 8 tor stps @ **66** c/c near joint for the height of **584**

SEISMIC ANALYSIS

PARAMETER	STAGING		
	12 M		
DEFLECTION	0.01628		

A) TANK FULL

1. W= (339741+(86423/3)) Kg
   = 368548

2. d AT TOP= (368548/10) X 0.01628 m
   = 0.6000

3. PERIOD T= 2 X 3.14 X SQRT(d/9.81) sec
   = 1.5537

Refer Clause 6.4.5 of IS 1893 for medium strata For T > 0.55

4. Sa/g= **1.67/T =** 1.0749

5. ah=	(Z/2) (I/R) (Sa/g)	z=	0.16
=	0.042996	I=	1.5
=	0.042996	R=	3

6. SEISMIC FORCE = W X ah Kg
   = **15846**

WT OF WATER 198888

B) TANK EMPTY

1. W= (368548-198888) Kg
   = 169660

2. d AT TOP= (169660/368548)*0.6m
   = 0.2762

3. PERIOD T= 2 X 3.14 X SQRT(d/9.81) sec
   = 1.0543

REFER FIG 2, IS - 1893/1984, 5 % CRITICAL DAMPING

4. Sa/g= **1.67/T =** 1.584

5. ah=	(Z/2) (I/R) (Sa/g)	z=	0.16
=	0.06336	I=	1.5
=	0.06336	R=	3

6. SEISMIC FORCE = W X ah Kg
   = **10750**

Wind Analysis: (IS 875/1987, Pt III)

Design wind speed at any height Z m/sec

$V_z = V_b \times K_1 \times K_2 \times K_3$

Where Vb = Basic wind pressure = 39.0 m/sec
K1 = Probability factor (risk factor) = 1.0
K2 = Terrain height and structure size facore
(Class A, Category 3, Ht = 14.250) = 0.961
K3 = 1.0 (Topography factor)

Hence Vz = 37.48 m/sec

Design wind pressure Pz = 0.6 X 37.479 ^2 = 843 N/m^2 84.3 Kg/m^2

From appendix D, Reynold's number

$$R = \frac{D\,V_d}{Y} = \frac{8.21 \times 37.48}{1.46 \times 10^{-5}} = 21{,}075{,}520$$

and E/D = 0.001

$$\text{From fig (5),}\quad \frac{C_f}{1 + 2\,E/D} = 0.81 \qquad \text{Hence } C_f = 0.8116$$

Hence wind on container = Cf Ae pd
= 0.8116 X 8.21 X 5.2 X 84 = 2920 Kg

Column :

$$R = \frac{D\,V_d}{Y} = \frac{0.5 \times 37.48}{1.46 \times 10^{-5}} = 1{,}283{,}527$$

and E/D = 0.001

$$\text{Hence}\quad \frac{C_f}{1 + 2\,E/D} = 0.9 \qquad \text{Hence } C_f = 0.9018$$

Hence wind on column = Cf Ae pd
= 0.9018 X 2.5 X 11.3 X 84.281
= 2147 Kg

$$\text{Hence equivalent wind at top} = \frac{2147}{2} = 1074 \text{ Kg}$$

Length of brace = 2 X 3.505 + (3.505 X 2X2)X0.9
= 19.628 m

$$\textbf{Braces :} \quad = \frac{2 \times 1.3 \times 0.4 \times 19.6 \times 84.3}{2} = 863.51 \text{ Kg}$$

Total wind force at top = 2920 + 1074 + 864
= 4857 Kg

Hence Horizontal design force = 15846 kg

PARAMETER		STAGING		
		12 M		
a) HORZ DESIGN FORCE (Kg)	Seismic	15846		
ALLOWABLE STRESSES CAN BE INCREASED BY 1.333				
b) Mult Factor		15846/10000		
		1.5846		
REFER APPENDIX -I				
Max BM in Periphery Column	1.5846 X			
@ Bot Kg-m		**5074**		
M1=		8040		
@ Top Kg-m		**-1643**		
M2=		-2604		
Max BM in Interior Column	1.5846 X			
@ Bot Kg-m		**5878**		
M3=		9314		
@ Top Kg-m		**3892**		
M4=		6167		
	1.5846 X			
Additional Axial Load in Col Kg		**5632**		
		8925		
			Area of periphery col	1963
Vert Seismic force on end col	Area of peri col x vert seismic force / Total area of cols		area of 4 nos of peri col	7854
Force/peri Col ' Kg' =		1585	Area of int	1963
	Area of int col x vert seismic force / Total area of cols		Area of int	1963
Force/int Col ' Kg' =		1585	total	9817
Max BM in Brace at GL Kg - m	1.5846 X			
		-2797		
=		-4432		
		-2704		
=		-4285		
Max BM in Brace at other lv	1.5846 X			
		-4470		
=		-7083		
		-4425		
=		-7012		

DESIGN OF PERIPHERY COLUMN : **Cast in M-250, Design in M-250**

DIA OF COL 'd' mm = **500** Area of Column 'Ac' cm^2= 1963

PARAMETER STAGING

12 M

a) WT OF CONTAINER 70130

b) NET WT OF CONT 67157

c) WT OF STAGING 20689

c1) Net WT OF STAGING 18335

d) AXIAL LOAD/COL'Kg' WT OF CONT + WT OF STAGING

= 90819 non Seismic

e) Ac Required ' cm^2' = Axial Load /74.7

= 1216 **OK**

f) As Required 'cm^2' = Ac x 0.008

= 9.726

g)As Provided

Dia of Bar (TOR) 16

No of Bars 10

Area 'cm^2' "Asp" 20.11

Links : 8 tor rings at 150 mm c/c

h)Area of Compo Sec

Ag=Ac+(1.5x11-1)xAsp 2275

I=3.14x(d^4)/64+.5x15.5Aspx(d/2-4.6)^2

= 370378

Z=I x 2/d = 14815

INTERACTION CHECK

Axial Load Seismic

'Ph' = Net wt of CONTAINER + Additional Axial Load due to Horz Force+Vert Seismic Force "Kg"

= 94416

6o,cal= Ph/Ag " Kg/cm^2"

41.50

R(Shear in col)=(M1+M2)/(LA) , LA "m"= 2

R= 2718.40

Depth of Footing = 0.4 m

Net BM =M1-(R x0.4/3) Kg-m

Net BM = 7677.85

6b,cal=Net BM/Z "Kg/CM^2"

51.82

CHECK = (6o,cal/60)+(6b,cal/85) < 1.3333333

(6o,cal/60)+(6b,cal/85)= **1.3014**

DESIGN OF INTERIOR COLUMN : **Cast in M-250, Design in M-250**

DIA OF COL 'd' mm = 500 Area of Column 'Ac' cm^2= 1963

PARAMETER STAGING

12

a) WT OF CONTAINER 71869

b) NET WT OF CONT 71869

c) WT OF STAGING 13083

d) AXIAL LOAD/COL'Kg' WT OF CONT + WT OF STAGING

= 84952 non Seismic

e) Ac Required ' cm^2' = Axial Load /74.7

= 1137.24

f) As Required 'cm^2' = Ac x 0.008

= 9.098

g)As Provided

Dia of Bar (TOR) 16

No of Bars 10

Area 'cm^2' "Asp" 20.11

Links : 8 tor rings at 150 mm c/c

h)Area of Compo Sec

Ag=Ac+(1.5x11-1)xAsp 2275.14

I=3.14x(d^4)/64+.5x15.5Aspx(d/2-4.6)^2

= 370378

Z=I x 2/d = 14815

INTERACTION CHECK

Axial Load Seismic

'Ph' = Net wt of CONTAINER +Vert Seismic Force "Kg"

= 84952

6o,cal= Ph/Ag " Kg/cm^2"

37.34

R(Shear in col)=(M3+M4)/(LA) , LA "m"= 3.767

R= 4109.80

Depth of Footing = 0.4 m

Net BM =M3-(R x 0.4/3) Kg-m

Net BM = 8766.35

6b,cal=Net BM/Z "Kg/CM^2"

59.17

CHECK = (6o,cal/60)+(6b,cal/85) < 1.3333333

(6o,cal/60)+(6b,cal/85)= **1.3185**

CHECK OF COLUMN BY LIMIT STATE METHOD

FOR INTERIOR COLUMN REFER SP :16/1980 DESIGN AIDS TO IS 456/2000

D = 500 d'/D = 46/500 0.092

fck = 25 fy = 415

condition	non Seismic		Seismic
P	850 KN		850 KN
Factored load Pu= 1.5P	1274 KN	Pu= 1.2 P	1019 KN
BM	----		87.66 KNm
Factored BM	----	Bmu=1.2M	105.20 KNm
$\frac{Pu}{fck\ D^2}$ =	0.2039		0.1631
$\frac{Mu}{fck\ D^2}$	----		0.03
$\frac{P}{fck}$ (Chart 56)			Negligible

Mini concrete area= $\frac{12743}{12.1319}$ 1050 cm^2 < 1963 O K

Provide **10 X 16** tor main bars

FOR PERIPHERY COLUMN

D = 500 d'/D = 46/500 0.092

fck = 25 fy = 415

condition	non Seismic		Seismic
P	908 KN		944 KN
Factored load Pu= 1.5P	1362	Pu= 1.2 P	1133 KN
BM	----		76.78 KNm
Factored BM	----	Bmu=1.2M	92.13 KNm
$\frac{Pu}{fck\ D^2}$ =	0.2180		0.1813
$\frac{Mu}{fck\ D^2}$	----		0.03
$\frac{P}{fck}$ (Chart 56)			Negligible

Mini concrete area= $\frac{13623}{12.1319}$ 1122.901 cm^2 < 1963 O K

Provide **10 X 16** tor main bars

Design of Brace : 300 x 400 , M250 conc

B= 300 D= 400

Span = **4.005** m A = **0.12** Sq m

M = **7083** Kgm M = **7012** Kgm

$$R = \frac{7083 + 7012}{4.005} = 3519 \text{ Kg}$$

Self wt = 300 Kg m

$$SF = 3519.4 + \frac{300 \times 4.005}{2} = 4120 \text{ Kg}$$

$$\text{Net BM} = 7083.2 + \frac{300 \times 4.005^2}{12} - \frac{4120.124 \times \times 0.5}{3}$$

$$= 7083.2 + 401.00063 - 687$$

$$= 6798 \text{ Kg m}$$

$$As = \frac{6797.5 \times 100}{2300 \times 32.6} = 10.56 \text{ Sq cm}$$

Provide at top and bottom 3x 16 tor str 6.032 sqcm

Extra at top and bottom near support upto 1000 2x 20 tor

TOTAL

$$\text{Tau v} = \frac{4120.124}{0.3 \times 36.0} = 3.76267012 \text{ Kg/cm2}$$

$$\frac{100\,As}{bd} = 1.140 \qquad \text{Tau c} = 4.381 \text{ Kg/sqcm}$$

Provide stp : Supp to 700, 8 tor @ 100 c/c
700 to centre, 8 tor @ 150 c/c

APPENDIX - F

Footing for column as per Pg 410 of Jain & Jaikrishna									
FOOTING FOR INTERIOR COLUMN, 4 Nos				SBC		**15 T/m^2**			
GRADE OF C CAST IN M-250, DESIGN IN M-250						**500 dia**			
Max axial load =	**84952**	**kg**				**Provide pedestal of 150 x150 all around column**			
SBC =	15	(T/m^2)	Col size	b =		0.650 m	d =	0.650	11.14 Q =
Axial Load:	84953	Kg	A fact=	1.1					1.5 k=
Area req=	6.229886667	m^2	Size req =	2.4960	beq	0.89375			
Provide sq foot of size =	**2.6**	m, A pro=		6.76	> A req OK Else		a=	2.6	
Design pressure =	12567.01	Kg/m^2 < SBC OK					c=	2.6	
SF=	31857.375	Kg	Offset =	0.975	m				
BM=	15530.47031	Kg-m	Cover	5	cm at ends				
d req =	39.49496025	cm	D=	44.99496		DIA		10	
Provide footing of depth	**55**	cm red to		**30**	cm at ends				
As =	15.46936632	cm^2	As req	19.69616	x		**10** tor		
So =	34.75413189	cm	As req	11.06258	x		**10** tor		
Asmin=	17.16	cm^2	As req	21.84874	x		**10** tor		
Provide bothways at bottom	21.84874		**Say 22 x**	**10**	tor				
Provide	**2600**	**x**	**2600**	**x**		**550 /300Thk footing**			

Footing for column as per Pg 410 of Jain & Jaikrishna									
FOOTING FOR PERIPHERIAL COLUMN, 4 Nos						**500 dia**			
GRADE OF C CAST IN M-250, DESIGN IN M-250						**Provide pedestal of 150 x150 all around column**			
Max axial load =	**90819**	kg							
SBC =	15	(T/m^2)	Col size	b =		0.650 m	d =	0.650	11.14 Q =
Axial Load:	90820	Kg	A fact=	1.1					1.5 k=
Area req=	6.660133333	m^2	Size req =	2.5807	beq	0.89375			
Provide sq foot of size =	**2.6**	m, A pro=		6.76	> A req OK Else		a=	2.6	
Design pressure =	13434.91	Kg/m^2 < SBC OK					c=	2.6	
SF=	34057.5	Kg	Offset =	0.975	m				
BM=	16603.03125	Kg-m	Cover	5	cm at ends				
d req =	40.83598822	cm	D=	46.33599		DIA		10	
Provide footing of depth	**55**	cm red to		**30**	cm at ends				
As =	16.53770731	cm^2	As req	21.05641	x		**10** tor		
So =	37.1543119	cm	As req	11.82658	x		**10** tor		
Asmin=	17.16	cm^2	As req	21.84874	x		**10** tor		
Provide bothways at bottom	21.84874		**Say 22 x**	**10**	tor				
Provide	**2600**	**x**	**2600**	**x**		**550 /300Thk footing**			

Footing for column as per Pg 410 of Jain & Jaikrishna									
FOOTING FOR INTERIOR COLUMN, 4 Nos				SBC		20 T/m^2			
GRADE OF C CAST IN M-250, DESIGN IN M-250						500 dia			
Max axial load =	84952	kg				Provide pedestal of 150 x150 all around column			
SBC =	20	(T/m^2)	Col size	b =		0.650 m	d =	0.650	11.14 Q =
Axial Load:	84953	Kg	A fact=	1.1					1.5 k=
Area req=	4.672415	m^2	Size req =	2.1616	beq	0.85625			
Provide sq foot of size =	**2.3**	m, A pro=		5.29	> A req OK Else		a=	2.3	
Design pressure =	16059.17	Kg/m^2 < SBC OK					c=	2.3	
SF=	30472.27174	Kg	Offset =	0.825	m				
BM=	12569.81209	Kg-m	Cover	5	cm at ends				
d req =	36.30124375	cm	D=	41.80124		DIA		10	
Provide footing of depth	**55**	cm red to		**30**	cm at ends				
As =	12.52035668	cm^2	As req	15.94138	x		**10** tor		
So =	33.24308268	cm	As req	10.5816	x		**10** tor		
Asmin=	15.36	cm^2	As req	19.55691	x		**10** tor		
Provide bothways at bottom	19.55691		**Say 20 x**	**10**	tor				
Provide	**2300**	**x**	**2300**	**x**		**550 /300Thk footing**			

Footing for column as per Pg 410 of Jain & Jaikrishna
FOOTING FOR PERIPHERIAL COLUMN, 4 Nos **500 dia**
GRADE OF C CAST IN M-250, DESIGN IN M-250 **Provide pedestal of 150 x150 all around column**
Max axial load = 90819 kg
SBC = 20 (T/m^2) Col size b = 0.650 m d = 0.650 11.14 Q =
Axial Load: 90820 Kg A fact= 1.1 1.5 k=
Area req= 4.9951 m^2 Size req = 2.2350 beq 0.85625
Provide sq foot of size = **2.3** m, A pro= 5.29 > A req OK Else a= 2.3
Design pressure = 17168.24 Kg/m^2 < SBC OK c= 2.3
SF= 32576.73913 Kg Offset = 0.825 m
BM= 13437.90489 Kg-m Cover 5 cm at ends
d req = 37.53383097 cm D= 43.03383 DIA **10**
Provide footing of depth **55** cm red to **30** cm at ends
As = 13.38503401 cm^2 As req 17.04231 x **10** tor
So = 35.53890703 cm As req 11.31238 x **10** tor
Asmin= 15.36 cm^2 As req 19.55691 x **10** tor
Provide bothways at bottom 19.55691 **Say 20 x** **10** tor
Provide 2300 x 2300 x 550 /300Thk footing

Footing for column as per Pg 410 of Jain & Jaikrishna
FOOTING FOR INTERIOR COLUMN, 4 Nos **SBC** **10 T/m^2**
GRADE OF C CAST IN M-250, DESIGN IN M-250 **500 dia**
Max axial load = 84952 kg **Provide pedestal of 150 x150 all around column**
SBC = 10 (T/m^2) Col size b = 0.650 m d = 0.650 11.14 Q =
Axial Load: 84953 Kg A fact= 1.1 1.5 k=
Area req= 9.34483 m^2 Size req = 3.0569 beq 0.96875
Provide sq foot of size = **3.2** m, A pro= 10.24 > A req OK Else a= 3.2
Design pressure = 8296.191 Kg/m^2 < SBC OK c= 3.2
SF= 33848.46094 Kg Offset = 1.275 m
BM= 21578.39385 Kg-m Cover 5 cm at ends
d req = 44.71581025 cm D= 50.21581 DIA **10**
Provide footing of depth **60** cm red to **30** cm at ends
As = 19.48475674 cm^2 As req 24.8087 x **10** tor
So = 33.47521232 cm As req 10.65549 x **10** tor
Asmin= 21.915 cm^2 As req 27.90298 x **10** tor
Provide bothways at bottom 27.90298 **Say 28 x** **10** tor
Provide 3200 x 3200 x 600 /300Thk footing

Footing for column as per Pg 410 of Jain & Jaikrishna
FOOTING FOR PERIPHERIAL COLUMN, 4 Nos **500 dia**
GRADE OF C CAST IN M-250, DESIGN IN M-250 **Provide pedestal of 150 x150 all around column**
Max axial load = 90819 kg
SBC = 10 (T/m^2) Col size b = 0.650 m d = 0.650 11.14 Q =
Axial Load: 90820 Kg A fact= 1.1 1.5 k=
Area req= 9.9902 m^2 Size req = 3.1607 beq 0.96875
Provide sq foot of size = **3.2** m, A pro= 10.24 > A req OK Else a= 3.2
Design pressure = 8869.141 Kg/m^2 < SBC OK c= 3.2
SF= 36186.09375 Kg Offset = 1.275 m
BM= 23068.63477 Kg-m Cover 5 cm at ends
d req = 46.23410909 cm D= 51.73411 DIA **10**
Provide footing of depth **60** cm red to **30** cm at ends
As = 20.83040748 cm^2 As req 26.52204 x **10** tor
So = 35.78706794 cm As req 11.39138 x **10** tor
Asmin= 21.915 cm^2 As req 27.90298 x **10** tor
Provide bothways at bottom 27.90298 **Say 28 x** **10** tor
Provide 3200 x 3200 x 600 /300Thk footing

DESIGN OF RAFT FOUNDATION M-250 SBC **7.5** t/m^2

Wt of ESR = 352390.9 + 95839 = 448230

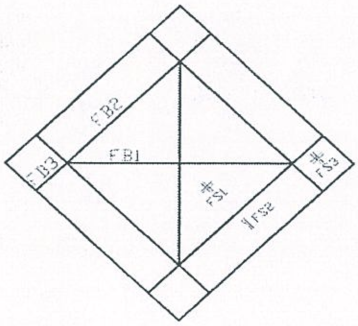
Area reqd = $\dfrac{448229.9 \times 1.15}{7500}$ = 68.73 m ^2

Provide raft as shown

Size of Raft = 8.3 say 8.4 m

Area of raft provided = 1 X 8.4^2 = 70.56 > 68.73
hence O K

p= 6353 kg/m^2



Span of FB1= 4.005
Span of FB2= 5.664
Span of FB3= 1.368
Panel of FS1= 2.832 x 5.664
Span of FS2= 1.368

SLAB FS1 250 THK size 2.832 X 5.664
Provide haunch of 100 x 300 at top near FB2

Dia of inscribed circle = 5.664/(1+(2)^0.5) = 2.35

- Ve BM = (6353x 2.832^2)/30 = 1699 Kgm

d req = SQRT (1699/ 11.14) = 13 say 16 cm
Dreq = 220 mm < 250 mm OK

As = $\dfrac{1699 \times 100}{2070 \times 18.5}$ = 4.44 cm^2

Pro at top & bottom 10 tor @ 100 c/c bothways in SQ Mesh OK 7.853982

0 c/c
7.854 c/c

SLAB FS2 Span = 1.368 m THK 350 Reducing to 200 at end

Cantilever span = 1.368 - 0.15 = 1.218 m clear

BM = $\dfrac{6353 \times 1.218^2}{2}$ = 4713 kgm

d req = SQRT (4713/ 11.14) = 21 say 21 cm
Dreq = 220 mm < 350 mm OK

Provide 350 depth reducing to 200 at ends

As = $\dfrac{4713 \times 100}{2070 \quad 29.5}$ = 7.72 cm^2

provide 10 tor @ 100 c/c 7.853982
+0 tor @ 175 c/c at bottom str bars OK
Dist- 8 X 8 tor dist bars at bottom

SLAB FS3 Panel 1.368 x 1.368 m 225

Provide 8 X 8 tor bothways at bottom
Provide 3 X 12 tor diagonal bars at bottom

RAFT BEAM FB1 **300 X 700** UPSTAND

SPAN = 4.005

Loading = $6353 \times \frac{5.664^2}{6}$ = 33968 Kg Triangular

SF = 33968 /2 = 16985

-ve BM = $\frac{5}{48} \times 33968.32 \times 4.005 - 16985 \times 0.5 / 3$

= 14171.2 - 2831 = 11341 Kgm

+ve BM = 0.6 X 14171 = 8503 Kgm

d req = SQRT (11341/ 11.14 x 0.3) = 59 say 79 cm
Dreq = 800 mm REVISE

-ve As = $\frac{11341 \times 100}{2070 \times 61.75}$ = **8.87** cm^2

+ve As = $\frac{8503 \times 100}{2070 \times 61.75}$ = **6.65** cm^2

Provide at top 2 X 16 tor str
2 X 16 tor extra in centre piece of length 2805

Provide at bottom 2 X 16 tor str
2 X 16 tor + 1 X 12 tor extra near supp upto 1000

$\frac{100 As}{bd} = \frac{9.17 \times 100}{30 \times 61.75}$ = 0.495

Hence Tau c = **3.085** Kg/cm2

Net SF = 11270 Kg

8 tor stp = $\frac{1.00531 \times 2300 \times 61.75}{11270}$ = 12.67 say 125

Provide stp : Supp to 1000 8 tor @ 125 c/c
1000 to centre, 8 tor @ 200 c/c

RAFT BEAM FB2 300 X 900 UPSTAND
Span = 5.664

loading

FS1 = $\frac{6353 \times 5.664^2}{12}$ = 16985 Kg Triangular

FS2 = 6353 X 1.368 = 8691 Kg/m

SF = $\dfrac{16985 + 8691 \times 4.005}{2}$ = 25897 Kg

-ve BM = $\dfrac{5}{48} \times 16985 \times 5.664 - \dfrac{8691 \times 5.664^{2}}{12} - 25897 \times 0.5 / 3$

-ve BM = 10021.2 + 23235 - 4316.16 = 28940 Kg m

+ve BM = 0.6 X 10021.15 + 0.5 x23235 = 17630 Kgm

-ve As = $\dfrac{28940 \times 100}{2070 \times 81.75}$ = 17.10 cm^{2}

+ve As = $\dfrac{17630 \times 100}{2070 \times 81.75}$ = 10.42 cm^{2}

Mini As = $\dfrac{0.85}{415} \times 30 \times 81.75$ = 5.02 cm^{2}

provide at top 2 X 20 tor str
2 X 20 tor extra in centre piece of length .7 l

Provide at bottom 2 X 20 tor str
3 X 20 tor + 1 X 16 tor extra near supp upto l/4

$\dfrac{100\,As}{bd} = \dfrac{17.71 \times 100}{30 \times 81.75}$ = 0.722

Tau c = 3.545 Kg/cm^{2}

Net SF = 17203 Kg

8 tor stp = $\dfrac{1.00531 \times 2300 \times 81.75}{17203}$ = 10.99 say 100

Provide Stps- Supp to 1200 8 tor @ 100 c/c Req
1200 to centre. 8 tor @ 200 c/c 30 X
Provide 1 X 12 tor str skin bars on each vertical face 2.261 OK 1.95 X

RAFT BEAM FB3 300 X 900 UPSTAND
Span = 1.368

Provide at bottom 2 X 20 tor str + 3 X 20 tor str + + 1 X 16 tor str
Provide at top 2 X 20 tor str

Stps 8 tor @ 100 c/c
Provide 1 X 12 tor str skin bars on each vertical face 2.261 OK

STAIRCASE : M250

Total Rise = 3.766666667

span = 5664

From GL to LDL total F 12 m
Rise per flight = 3.7667 m

As per NIT
1) Width of Flight = 1000 mm
2) Flight of Rise = 3.7667 m
3) no of risers = 23 nos
4) Rise = 163.769 mm
5) No of treads = 22 nos
6) Tread = 257.455

span = 5664 m

$$\text{Tread} = \frac{5664}{22} = 257.455 \text{ m}$$

WAIST SLAB : 160 THK 1000 wide span = 5664

$$\text{Load self} = \frac{400}{0.8438} = 475 \text{ Kg/m}^2$$

$$\text{Steps} = \frac{0.163769}{2} \times 2500 = 205 \text{ Kg/m}^2$$

LL = 300 Kg/m^2

Finish = 0 Kg/m^2

----

980 Kg/m^2

$$SF = \frac{980 \times 5.664}{2} = 2776 \text{ Kg}$$

$$BM = \frac{980 \times 5.664^2}{12} = 2620 \text{ Kgm}$$

d = 160 - 25 - 5= 130 mm

Mu = 2620 X 1.5 = 3930 Kgm

As = 10.47 cm2/m
So As total= 10.47 cm^2/1m

Provide **10 X** **12 tor** str at top and bottom in 1 m 11.310 OK

Dist- **8 tor @** **250 c/c** str at top and bottom

CANTILEVER BEAM: 300 X 400 span 1

Cantilever span 1 m

Load Self =			300 Kg/m^2
Stair =	2776		2776 Kg/m^2

			3076 Kg/m^2
SF =	3076 Kg		

$$BM = \frac{3076 \times 1^2}{2} = 1538 \text{ Kgm}$$

Mu =	2307 Kgm	
d =	40 - 2.5 + 1.0 =	36.5
As =	2.19 cm2	

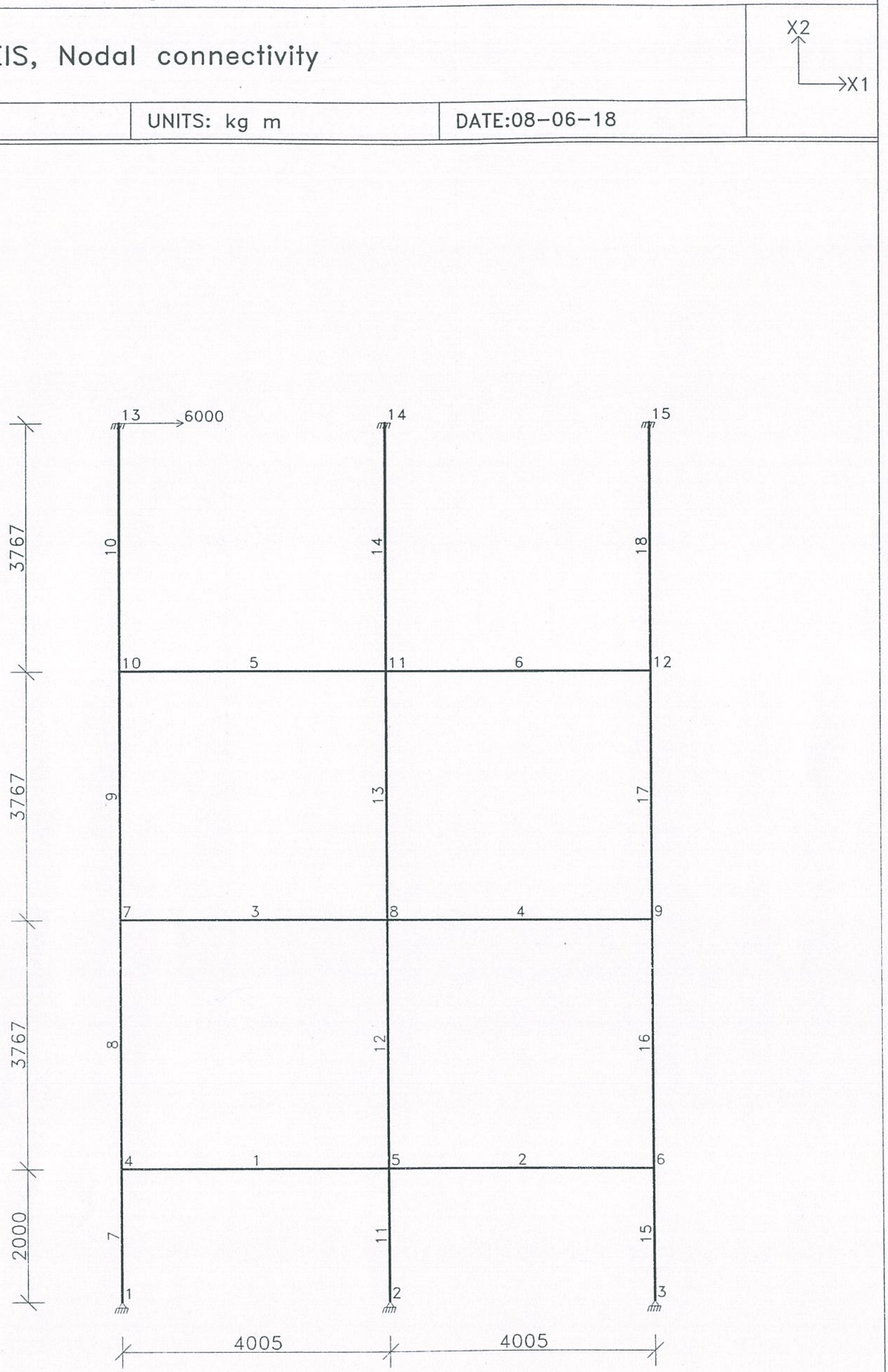
Provide at top	3x	16 tor str	12.316 sqcm
	2x	20 tor str	
At bottom	3x	16 tor str	12.316 sqcm
	2x	20 tor str	

stps - 8 tor @ 100 c/c

1.75 lakh - 12m stg -M P

Load 1: SEIS, Nodal connectivity

SCALE = 1:80	UNITS: kg m	DATE:08-06-18



1.75 lakh - 12m stg -M P
Deflection & Member End forces
Prepared by: MINAL

STATIC DEFLECTIONS for load no. 1 (Units: meter)
SEIS

Node	X1	X2	X6
1	0.00000	0.00000	0.0000000
2	0.00000	0.00000	0.0000000
3	0.00000	0.00000	0.0000000
4	0.00104	0.00002	-0.0008756
5	0.00104	0.00000	-0.0007925
6	0.00104	-0.00002	-0.0008756
7	0.00650	0.00006	-0.0014803
8	0.00650	0.00000	-0.0012803
9	0.00650	-0.00006	-0.0014803
10	0.01272	0.00007	-0.0013113
11	0.01272	0.00000	-0.0011280
12	0.01272	-0.00007	-0.0013113
13	0.01628	0.00007	0.0000000
14	0.01628	0.00000	0.0000000
15	0.01628	-0.00007	0.0000000
MAX.	0.01628	0.00007	-0.0014803
NODE	13	13	7

BEAM RESULTS for load no. 1 (Units: kg, kg*meter)
SEIS

Bm.	Node	Axial	V2	M3
1	4	-11.941 T	-1373.478	-2797.055
	5	11.941 T	1373.478	-2703.724
2	5	11.941 C	-1373.478	-2703.724
	6	-11.941 C	1373.478	-2797.055
3	7	-101.847 T	-2265.673	-4649.402
	8	101.847 T	2265.673	-4424.620
4	8	101.847 C	-2265.673	-4424.620
	9	-101.847 C	2265.673	-4649.402
5	10	205.324 C	-1992.974	-4093.914
	11	-205.324 C	1992.974	-3887.947
6	11	-205.324 T	-1992.974	-3887.947
	12	205.324 T	1992.974	-4093.914
7	1	-5632.125 T	1715.326	5073.206
	4	5632.125 T	-1715.326	-1642.555
8	4	-4258.647 T	1703.384	4439.610
	7	4258.647 T	-1703.384	1977.039
9	7	-1992.974 T	1601.538	2672.363
	10	1992.974 T	-1601.538	3360.629
10	10	0.000 C	1806.862	733.284
	13	0.000 C	-1806.862	6073.164
11	2	0.000	2569.349	5608.681
	5	0.000	-2569.349	-469.984
12	5	0.000	2593.231	5877.433
	8	0.000	-2593.231	3891.270
13	8	0.000	2796.925	4957.970
	11	0.000	-2796.925	5578.046

BEAM RESULTS for load no. 1 (Units: kg, kg*meter)
SEIS

Bm.	Node	Axial	V2	M3
14	11	0.000 C	2386.277	2197.847
	14	0.000 C	-2386.277	6791.257
15	3	5632.125 C	1715.326	5073.206
	6	-5632.125 C	-1715.326	-1642.555
16	6	4258.647 C	1703.384	4439.610
	9	-4258.647 C	-1703.384	1977.039
17	9	1992.974 C	1601.538	2672.363
	12	-1992.974 C	-1601.538	3360.629
18	12	0.000	1806.862	733.284
	15	0.000	-1806.862	6073.164
MAXIMUM		5632.125 C	2796.925	-6791.257
Beam no.		15	13	14

Venkat
31/8/18
N Dindorkar

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