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-1

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.

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Maulana Azad National Institute of Technology
भोपाल (म. प्र.)/BHOPAL-462007 (M.P.)

1.50 LAKH LIT CAPACITY RCC ESR OF 12 M STG HT FOR MADHYA MRADESH 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
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email : aquades@rediffmail.com

Data :

Capacity	:	150 m^3
FSL	:	17.5 m
LDL	:	12.0
GL	:	0.0
F L	:	-2.5
Free Board	:	0.5 m
SBC of Soil	:	10 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 : SC/18023/ 17, 18 dt- 12 /6/2018

Madhya Pradesh_1.50st12

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

150,000 ltrs rcc esr on 12 m stg

CAPACITY	150	m^3
FSL	17.5	m
LDL	12	m
GL	0	m
FL	2.5	m
FREE BOARD	0.5	m
SBC	10	T/M^2
SEISMIC ZONE	III	

CAPACITY CALCULATIONS

SWD= 17.5-12 = 5.5 m

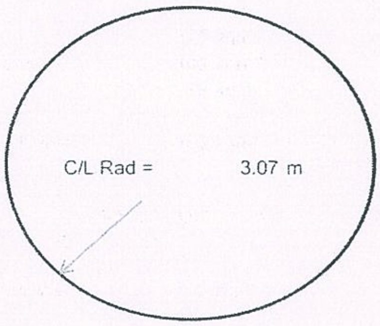
Area= 150/5.5 27.273 m^2

clear radius = 2.947 say = **2.95**

Assume wall thickness = 200 mm

Hence centre line radius = **3.07**

DESIGN OF ROOF SLAB **150** THK **M300** Concrete
PROVIDE CAMBER OF + **40 mm IN CENTER**



DL=	375	kg/m^2
Water proofing=	100	kg/m^2
Live load=	75	kg/m^2
Total=	550	kg/m^2

BM= $\frac{2}{16} \times 550 \times 3.07^2$ = 648 Kgm

Wt of central shaft = 500 Kg assumed

M_{total} = Total + Ve BM across diameter

$= \frac{PD}{2 \times p} \times \left(1 - \frac{2d}{3D}\right)$

where D= 6.14
dia of central shaft d= 1
wt of central shaft P= 500
p = 3.141593

$= \frac{500 \times 6.14}{2 \times p} \times \left(1 - \frac{2 \times 1}{3 \times 6.14}\right)$ = 436

Verified
3/8/18

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Mm = mean +ve BM across diameter

$$= \frac{Mtotal}{D} = \frac{436}{6.14} = 72 \text{ Kgm}$$

For restrained edges

$$\text{-ve BM at edge} = \frac{72}{3} = 24 \text{ Kgm}$$

Hence BM max = 672 Kgm

$$As = \frac{67200}{0.87 \times 1500 \times 10} = 5.15 \text{ cm}^2$$

$$\text{Mini As} = \left(0.3 - \frac{5}{350}\right) \times 15 \times 0.8 = 3.43 \text{ cm}^2 \quad \text{OR}$$

$$\text{Mini As} = 0.24 \times 15 = 3.6 \text{ cm}^2$$

Provide **8 tor @ 90 c/c** bothways in square mesh at bottom, all the bars to be brought back at top for a distance of 1430 mm As pro 558.5 mm^2 OK

Provide 4 X 10 tor Hoop bars at bottom near support

CHECK FOR VERTICAL DEFLECTION THICKNESS 150

As provided 8 tor @ 90 5.15 cm^2

$$\frac{100 As}{b d} = \frac{5.15}{15} = 0.344$$

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

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

Span = sqrt(pi() X 3.07^2) 5.442 m

$$\text{Hence depth required} = \frac{5.442}{39} = 0.139538 \text{ OK}$$

DESIGN OF WALL 200 THICK

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

Wall :
****

*C LRadius 'R' m :	3.07
*Thkof wall 'Tw' m :	0.2
*Htof Water 'h' m :	6
Hoop Ten 'HT' Kgm :	17700
Disp"HT*R/TwE" 'd' :	271695
Slope"d/h" 'Theta' :	45282.5
u "(3/(RTw)^2)^.25":	1.67956234
Z = "(ETw^3/12)" :	0.000666667
Mom Stiffness"2uZ" :	0.002239416
Cor Thrust"2 u^2 Z":	0.00376124
Thu Stiff "4 u^3 Z":	0.012634473

Base Slab :
****

*Thk of slab 'Ts' m:	0.3
Eff Width"6Ts" 'Be':	1.8
Thu Stif "EA/R1R2":	0.0849738
M St"EATs^2/12R1R2":	0.000637303

Bottom Ring Beam :
****
Size

* 'b' m:	0
* 'd' m:	0
Thu Stif EA/R^2 :	0
M St "EAd^2/12RR" :	0

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'-	45282.5	y'-	271695	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:	0.00287672	-0.0038	-920.5036
Eq2=> Sum of Thu=0:	-0.00376124	0.09761	3262.4047

From Eqation 1 & 2		
Invert of above	366.0611185	14.1058
matrix	14.10580804	10.7886

0'= -290941.7
y'= 22212.289

HT in wall = 1447.054676

HT in B Slab = 5794.510245

HT in BotRingBeam = 0

HT in Gallery = 0

BM in wall = 185.418178

BM in B Slab = -185.418178

BM in BotRingBeam = 0

BM in Gallery = 0

DESIGN OF WALL 200 THICK M300

Ht at Base = 1448 very small

$$6o = \frac{1448}{2000} = 0.724 \text{ Kg/cm}^2 < 13 \text{ OK}$$

Refer Appendix-I for hoop steel required as per Table 12 of IS 3370 and graph showing hoop steel provided and required.

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

Base to	420	8 tor	@ 60 c/c	8 nos	8.378 cm2
420 to	1770	8 tor	@ 50 c/c	27 nos	10.053 cm2
1770 to	2430	8 tor	@ 60 c/c	11 nos	8.378 cm2
2430 to	6000	8 tor	@ 70 c/c	51 nos	7.181 cm2

VERTICAL STEEL-

BM = 186 Kgm

Design BM = 186 Kgm

$$6bt = \frac{186 \text{ X } 6}{400} = 2.79 \text{ Kg/cm}^2$$

< 18 Hence OK

$$As = \frac{186 \text{ X } 100}{.87 \text{ X } 1500 \text{ X } 14.5} = 0.983 \text{ cm}^2$$

Mini As = 0.313 x 10 = 3.13 cm^2

Provide **8 tor @ 150c/c** on each face of wall

As pro 335.1

OK

Ref Apendix CC2 for crackwidth check for Hoop in wall. Wc = **0.1177 << 0.2 mm OK**

Ref Apendix CC3 for crackwidth check for Vert BM in wall. Wc = **-0.15 << 0.2 mm OK**

DESIGN FOR DIRECT TENSION IN WALL		**APPENDIX CC1**			
Tension =	172373	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	45	mm	**m=(280/3 6cbc) =**		9.333
Req As = 1.5* HT /(0.87 * fy*.9) =		795.70	**Lim W =**	0.2	
Provided Steel					
8 dia bars @	70 mm c/c	Cover=		49 mm (CHECK)	
I e As =	718.08 sqmm		ok		

Check Tensile Stress in Concrete

fct = (C Es Ast + HT)/(Ac+(m-1) Ast) = 1.05 M pa — Less Than 3 M Pa, OK

Calculation of Crack width

Apparent Strain e1 = T / As Es

e1 =	0.001200237
For design crackwidth = 0.2 mm	
e2 =	2 b d/ (3 Es As)
As such e2 =	0.000928404
For design crackwidth = 0.1 mm	
e2 =	b d / (Es As)
As such e2 =	0.0014 sqmm
For our case e2 =	0.0009
Avg Strain em = e1 - e2	
e m =	0.000271833
a cr =	144.33

Crackwidth = 3 a cr em = 0.1177

ok

CRACK WIDTH CALCULATIONS APP-CC2 VERT BM IN WALL OF ESR

DATA

Permissible crackwidth			0.2 mm	Ms =	0 kNm			
Width of the section		b	1000 mm	Mu=	0 kNm			
Overall Depth of the section		D	200 mm	Cover	0 mm			
Es			2.00E+05 Mpa					
fck			30 Mpa	Ec = 5000*sqrt(30)= 27386.13 Mpa		As req=	0	
Use	8 dia bars @ 200 mm c/c			*(For calculation of crackwidth use 0.5 Ec)*				
As =	251.50 sqmm per 1000mm	Fy =	415 Mpa			total steel	251.5	sqmm
a_{cr} =	96.08					Spacing	200	
d=(D-c-dia/2) =	196.00 mm	(CHECK)				Eq Dia	8.002746	cm
r =	0.001283172 , Modulus of elasticity m =		14.60593					
(x/d) = -mr+sqrt(mr(2+mr)) =	0.175771 ie x=		34.45 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 = 0.001124252

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 = 0.001686379

For our case e2 = 0.0011

A= (Ms/(Es*As*(d-(x/3)))*(h-x)/(d-x) = 0.00E+00

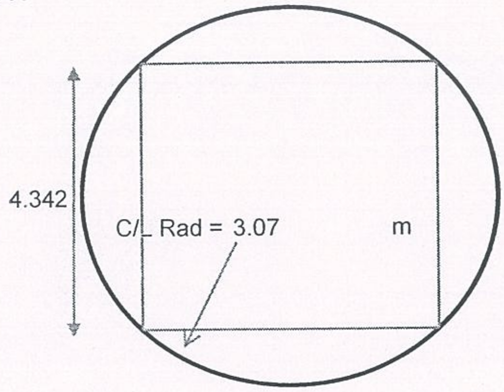
B= (1+2*(acr-c)/(h-x)) = 2.160726195

W=(A-e2)*3*acr/(B) = -0.15 OK

LA Z = d - (x/3) =	184.52	
Steel Tensile Stress fs = Ms/(z As) =	0.000	M Pa
Conc Comp Stress = 2Ms/(z b x) =	0.000	M Pa
CHECK STRESS LEVELS		
0.45 fcu =	13.5 M Pa	OK
0.8 fy =	332 M Pa	OK

BASE SLAB 300 THICK M300

SPAN = **4.342 X** **4.342**



Loading =

DL = 750

water = 6000

plaster = 42

6792 Kg/m^2

Refer table 22, IS 456/2000, Interior Panel

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

- ve BM = 0.032 X 6792 X 4.342^2 = 4098 Kgm

+ve BM = 0.024 X 6792 X 4.342^2 = 3074 Kgm

Net -ve BM at the face of support=

= 4098 - $\frac{(4098 + 3074) X 4 X (4.342 - 0.3) X 0.15}{4.342\text{^}2}$ = 3176 Kgm

SAY = 4098

6bt = $\frac{3176 X 6}{30\text{^}2}$ = 21.18 Kg/cm^2 <20 hence OK with steel contribution

-ve As = $\frac{1.5 x4098000}{0.8 x 0.87 x 415 x250}$ = 852 mm^2

+ve As = $\frac{1.5 x3074000}{0.8 x 0.87 x 415 x250}$ = 639 3m^2

Mini As = (0.3 - $\frac{20}{350}$) X 30 X 0.8 = 5.829 cm^2

Mini As = 0.313 x 15 = 469.5 mm^2

provide at bottom **10 tor @** **80 c/c** bothways in square mesh at bottom, alternate

bar bent up @ **870** from support 9.818 cm^2

provide at top **10 tor @** **160** bothways in sqmesh 4.909 cm^2

-ve As provided = 981.8 mm^2 OK

+ve As provided = 981.8 mm^2 OK

Hoop in base slab = $\frac{5795}{1500}$ = 3.87 cm^2

provide 2 X **12 tor** hoop bars at top & bottom near wall (As provided=4.523cm^2) Total 4 nos

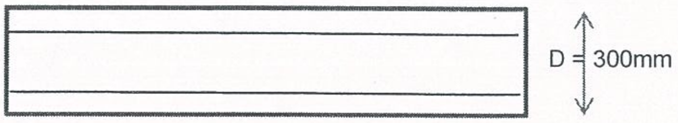
CHECK FOR CRACK WIDTH

Ref Appendix CC3, Wc = 0.1081672 << 0.2 mm OK

CHECK FOR NON CRACKING

Ast=78.544

Asb=61.3625



Area of steel provided at top= (9 - 1) X 981.8 = 78.544 cm^2

Area of steel provided at bottom= (1.5 X 9 -1) X 4.909= 61.3625 cm^2

As = (30) X 100 + 78.544 + 61.3625 = 3139.907 cm^2

$$y = \frac{3000 \times 15 + 78.544 \times 5.5 + 61.3625 \times 24.5}{3139.9065} = 14.95 \text{ cm}$$

$$I = \frac{100 \times 30^3}{12} + (3000)*(15 - 14.95)^2 + 78.544 \times 9.45^2 + 61.3625 \times 9.55^2$$

$$= 225000 + 7.5 + 7014.18 + 5596.413$$

$$= 237618 \text{ cm}^4$$

$$Z = 15894.18655 \text{ cm}^3$$

$$6bt = \frac{317600}{15894.18655} = 19.99 \text{ Kg/cm}^2 \quad < 20 \text{ hence OK}$$

CRACK WIDTH CAL APP-CC3 BM IN BASE SLAB OF ESR

DATA

Permissible crackwidth		0.2 mm	Ms =	40.98 kNm				
Width of the section	b	1000 mm	Mu=	61.47 kNm				
Overall Depth of the section	D	300 mm	Cover	45 mm				
Es		2.00E+05 Mpa						
fck		30 Mpa	Ec = 5000*sqrt(30)=	27386.13 Mpa	As req=	709.19		
Use 10 dia bars @ 80 mm c/c			*(For calculation of crackwidth use 0.5 Ec)*					
As = 981.80 sqmm per 1000mm	Fy =	415	Mpa				total steel	981.8 sqmm
a_{cr} = 59.03							Spacing	80 c/c
d=(D-c- 250.00 mm	(CHECK)						Eq Dia	10.00027 cm

r = 0.003927202 , Modulus of elasticity m = 14.60593

(x/d) = -mr+sqrt(mr(2+mr)) = 0.287 ie x= 71.75 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 0.000496156

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 0.000744234

For our 0.0005

A= (Ms/(Es*As*(d-(x/3)))*(h-x)/(d-x) = 1.18E-03

B= (1+2*(acr-c)/(h-x)) = 1.122946

W=(A-e2)*3*acr/(B) = 0.108167223 OK

LA,Z = d - 226.08

Steel Tensile Stress fs = 184.621 M Pa

Conc Comp Stress = 2N 5.053 M Pa

CHECK STRESS LEVELS

0.45 fcu	13.5 M Pa	OK
0.8 fy =	332 M Pa	OK

BASE BEAM 300 X 950 M300 span= 4.342 m

Triangular load = $\dfrac{6792 \times \pi \times 3.07^2}{4}$ = 50277 Kg

udl = 0.3 X 0.65 X 2500 = 487.5 Kg/m

SF = $\dfrac{50277 + 487.5 \times 4.342}{2}$ = 26197 Kg

-veBM = $\dfrac{5}{48} \times 50277 \times 4.342 + \dfrac{487.5 \times 4.342^2}{12} - \dfrac{26197 \times 0.5}{3}$

= 22740 + 766 - 4366

= 19140 Kgm

+veBM = 0.6 X 22740 + 0.5 X 766 = 14027 Kgm

-veAs = $\dfrac{1914000}{0.87 \times 1500 \times 89.5}$ = 16.39 cm^2

+veAs = $\dfrac{1402700}{0.87 \times 1900 \times 89.5}$ = 9.481 cm^2

As min = $\dfrac{0.85}{415} \times 30 \times 89.5$ = 5.5 cm^2

Provide at bottom 3 X 20 tor str
2 X 20 tor extra in centre piece of length 3040 15.71 Sqcm OK

Provide at top 3 X 20 tor str
1 X 16 tor str + 2 X 20 tor extra near supp upto 1085 17.72 Sqcm OK

Provide 2 X 10 tor str skin bars on each vert face 3.14 Sqcm OK

tau v = $\dfrac{26197}{30 \times 89.5}$ = 9.76 Kg/cm^2

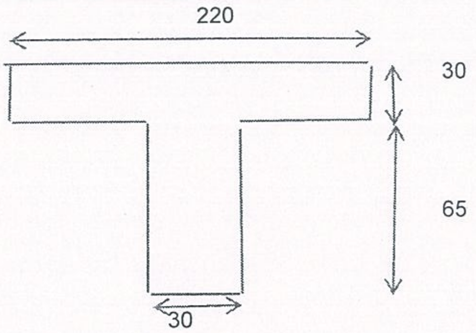
$\dfrac{100\ As}{b\ d} = \dfrac{17.7186 \times 100}{30 \times 89.5}$ = 0.660 Hence tau c = 3.42 Kg/cm^2

Net SF = 26197 - 3.42 X 30 X 89.5 = 17015 kg

8 tor stps @ $\dfrac{1.005 \times 1750 \times 89.5}{17015}$ = 9.26 say 90

supp to	1530	8 tor	stps @	90 c/c
1530	to centre	8 tor	stps @	200 c/c

NON CRACKING CHECK



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

$$= \frac{0.7 \times 4.342}{6} + 0.3 + 6 \times 0.3 = 2.607 \text{ m}$$

$$bf\ available = \frac{2.607}{2} + 3.07 \times (1 - .707) = 2.20301 \text{ m}$$

say 220 cm

$$A = 6600 + 1950 = 8550 \text{ cm}^2$$

$$Y = \frac{6600 \times 15 + 1950 \times 62.5}{8550} = 25.834 \text{ cm}$$

$$I = \frac{220}{12} \times 30\text{^}3 + 6600 \times 10.8\text{^}2 + \frac{30}{12} \times 65\text{^}3 + 1950 \times 36.67\text{^}2$$

$$= 495000 + 774679 + 686563 + 3E{+}06 = 4577813 \text{ cm}^4$$

$$Z = \frac{4577812.5}{25.834} = 177201.1 \text{ cm}^3$$

$$6bt = \frac{1402700}{177201.07} = 7.92 \text{ OK}$$

DESIGN OF GALLERY 120 THICK M300 Span = 1 m cantilever

DL = 300
LL = 300

Total = 600 Kg/m^2

BM = 300 m Kgm

$$As = \frac{30000}{0.87 \times 1900 \times 7.5} = 2.42\ cm^2$$

Provide at top 8 tor @ 200 c/c radial bars every ***alternate*** rod chima OK
Circumferential bars 4 x 8 tor at top & bot As pro 2.513

Check for wall as beam 200 wide X 6450 deep
It is tied at top by roof slab and at bottom by base slab

Roof Slab = 17363 Kg
Self Wt = 57868 Kg
Gallery = 13836 Kg

TOTAL 89067 Kg

$$WR = \frac{89067}{2 \times 3.1415} = 14176\ Kg$$

$$WR2\ (2a) = \frac{14176 \times 3.1416 \times 3.07}{2} \quad 68362$$

-ve BM = 0.137 X 68362 = 9366 Kgm
+ve BM = 0.07 X 68362 = 4786 Kgm
Torsion = 0.021 X 68362 = 1436

$$SF\ at\ support = 14176 \times \frac{3.141593}{4} = 11134\ Kg$$

$$SF\ at\ contra = 14176 \times \left(\frac{3.141593}{4} - 0.336 \right) = 6371\ Kg$$

$$6bt = \frac{936600 \times 6}{20 \times 645^2} = 0.6753921\ Kg/cm^2$$
< 18 hence OK

Clause 28, IS 456/1978, for deep beam

$$\frac{L}{D} = \frac{2 \times 3.142 \times 3.07}{4} \times \frac{1}{6.45} = \frac{4.83}{6.45} = 0.748 < 2$$

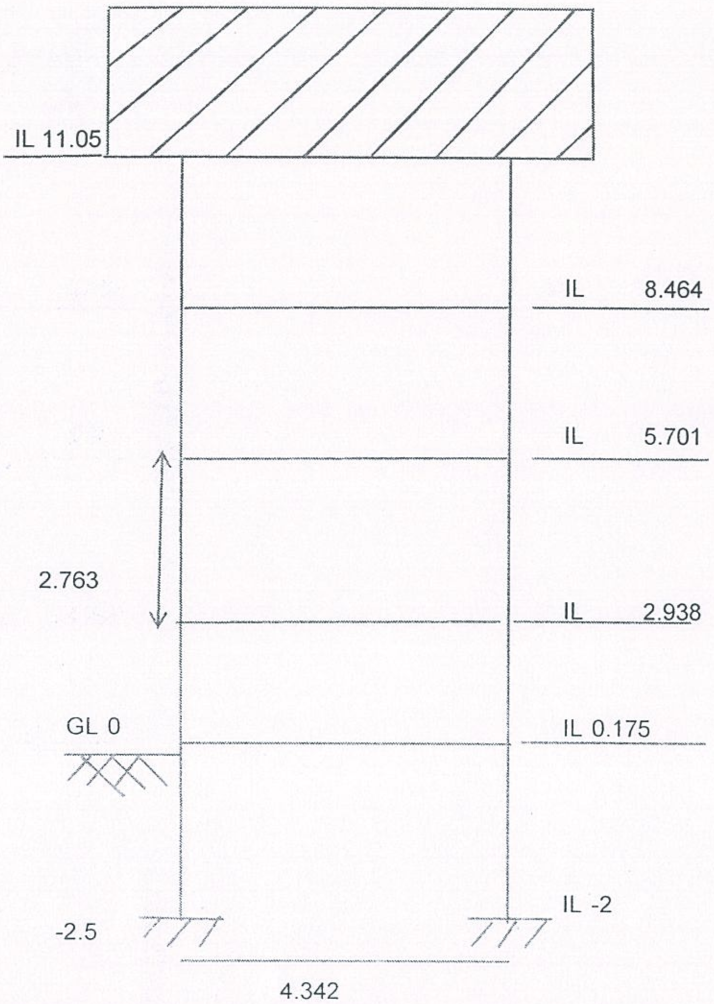
Hence Z = 0.2 X (4.83 + 2 X 6.5) = 3.546 m

$$As = \frac{936600}{1500 \times 354.6} = 1.77\ cm^2 \quad \text{very small}$$

$$Net\ SF = 6371 + \frac{1.6 \times 1436}{0.2} = 17859\ Kg$$

$$Tau\ v = \frac{17859}{20 \times 640} = 1.4\ Kg/cm^2$$
very small
Hence O K

Staging - consider 500 dia columns brace 250 350



COLUMN 500 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 **250 X 350** span= 4.342
no of br lvls= 4

$$A = 0.25 \times 0.35 = 0.0875 \text{ cm}^2$$

Assume an arbitrary force of 10 tonnes acting at cg of container

Force on one frame = 5000 kg
KINDLY NOTE AS CAPACITY IS LESS THAN 1000 KL AS PER CL 5.2 of IS 1893 (PART 25):2014
ONE MASS MODEL NEGLECTING CONVECTIVE MODE IS CONSIDERED

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

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

deflection= 0.01938 m

WEIGHT OF CONTAINER

Roof Slab	=	3.1416 X 3.17^2 X 550 =	17363
wall	=	2 X 3.1416 X 3.07 X 6.0 X2500 X 0.2 =	57868
Base Slab	=	3.1416 X 3.17^2 X 750 =	23677
Base Beam	=	0.195 X 4.342 X 2500 X 4 =	8467
Gallery	=	3.1416 X (4.17^2 - 3.17^2) X 600 =	13836
water	=	3.14159 X 2.95^2 X 6000 =	164038
plaster	=	(3.1416 X 2.96^2+2 X3.14X3 X 6) X 2080X0.02	5787

		TOTAL =	**291037** Kg

WEIGHT OF STAGING

column dia = 400 brace= 250 X 350 span= 4.342

staging ht= 12

Column	=	0.1963 X 4 X 13.55 X 2500 =	26605
Brace	=	4 X 4 X 0.0875 X 3.942 X 2500 =	13797

		TOTAL =	**40403** Kg

ADD FOR STAIR

MB1 = 8.0000 X 2217 = 17736

Total Stg wt = 40403 + 17736 = **58139** Kg

Deduct Live Load

Roof Slab	=	3.1416 X 3.17^2 X 75 =	2368
Gallery	=	3.1416 X (4.17^2 - 3.17^2) X 300 =	6918

		TOTAL =	**9286** Kg

Net Wt of Container= 291037 - 9286 = **281751** Kg

Deduct LL on Stair = $\frac{300}{920}$ X 58139 = 18959 Kg

Net Staging Load = 58139 - 18959 = **39180** Kg

SEISMIC ANALYSIS

PARAMETER	STAGING
	12 M
DEFLECTION	0.01938

A) TANK FULL

1. W= (281751+(39180/3)) Kg
   = 294811

2. d AT TOP= (294811/10) X 0.01938 m
   = 0.5713

3. PERIOD T= 2 X 3.14 X SQRT(d/9.81) sec
   = 1.5161

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

4. Sa/g= **1.67/T =** 1.1015

5. ah=	(Z/2) (I/R) (Sa/g)	z=	0.16
=	0.04406	I=	1.5
=	0.04406	R=	3

6. SEISMIC FORCE = W X ah Kg
   = **12989**

WT OF WATER 164038

B) TANK EMPTY

1. W= (294811-164039) Kg
   = 130773

2. d AT TOP= (130773/294811)*0.5714m
   = 0.2534

3. PERIOD T= 2 X 3.14 X SQRT(d/9.81) sec
   = 1.0099

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

4. Sa/g= **1.67/T =** 1.6536

5. ah=	(Z/2) (I/R) (Sa/g)	z=	0.16
=	0.066144	I=	1.5
=	0.066144	R=	3

6. SEISMIC FORCE = W X ah Kg
   = **8650**

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 = 15.550) = 0.974

K3 = 1.0 (Topography factor)

Hence Vz = 38.00 m/sec

Design wind pressure $P_z = 0.6 \times 38.002^2 = 866\ N/m^2 = 86.6\ Kg/m^2$

From appendix D, Reynold's number

$$R = \frac{D\ V_d}{Y} = \frac{6.34 \times 38.00}{1.46 \times 10^{-5}} = 16{,}502{,}065$$

and E/D = 0.001

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

Hence Cf = 0.81162

Hence wind on container = Cf Ae pd

= 0.81162 X 6.34 X 7.1 X 86.6

= 3166 Kg

Column :

$$R = \frac{D\ V_d}{Y} = \frac{0.5 \times 38.00}{1.46 \times 10^{-5}} = 1{,}301{,}425$$

and E/D = 0.001

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

Hence Cf = 0.9018

Hence wind on column = Cf Ae pd

= 0.9018 X 11.05 X 86.6 X 4 X 0.5

= 1727 Kg

$$\text{Hence equivalent wind at top} = \frac{1727}{2} = 863 \text{ Kg}$$

$$\textbf{Braces :} = \frac{3 \times 1.36 \times 0.35 \times 3.842 \times 2 \times 86.6}{2}$$

= 477 Kg

Total wind force at top = 3166 + 863 + 477

= 4506 Kg

Design of column-

Critical load is	seismic			
	12989			
a) HORZ DESIGN	seismic			
FORCE (Kg)	12989			

ALLOWABLE STRESSES CAN BE INCREASED BY 1.333

b) Mult Factor	12989/1000			
	1.2989			

REFER APPENDIX -I

Max BM in Col	1.2989 X			
@ Bot Kg-m	6857			
M1=	8907			
@ Top Kg-m	-1857			
M2=	-2412			
	1.2989 X			
Additional Axial	8460			
Load in Col Kg	10989			
Vert Seismic	Seismic force/ (2 X 4)			
Force/Col ' Kg' =	1624			
Max BM in Brace	1.2989 X			
Kg - m	-5497			
=	-7140			
Max Story Drift =	0.00565 X	1.298937266 =	0.007338996 m	
Panel =	2.763	P=	349176 Kg	
		Base Shear=	12989	

STABILITY INDEX = $\dfrac{\mathbf{0.00734 \times 349176}}{\mathbf{2.76300 \times 12989}}$ **=0.071402258** **<<< 1**

DESIGN OF COLUMN : 500 DIA SIZE, 4 Nos, Design in M250 Conc, Cast in M-250

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

PARAMETER STAGING

12 m

a) WT OF CONTAINER 291037

b) NET WT OF CONT 281751

c) WT OF STAGING 58139

d) AXIAL LOAD/COL'Kg' (Wt Of Cont + Wt Of Staging)/4
= 87294 Non seismic

e) Ac Required ' cm^2' = Axial Load/74.7
= 1168.59

f) As Required 'cm^2' = Ac x 0.008
= 9.349 0.476 OK

g)As Provided

Dia of Bar (TOR)	16
No of Bars	9
Area 'cm^2' "Asp"	18.10

Provide 8 TOR links at 150 c/c

h)Area of Compo Sec

Ag=Ac+(1.5x11-1)xAsp 2243.98

I=3.14x(d^4)/64+.5x15.5Aspx(d/2-4.8)^2
= 421243.76

Z=I x 2/d = 16849.75

INTERACTION CHECK

Axial Load seismic

'Ph' = Net wt of ESR + Additional Axial Load due to Horz Force+Vert Seismic Force "Kg"
= 97585

6o,cal= Ph/Ag " Kg/cm^2"
43.49

R(Shear in col)=(M1+M2)/(LA) LA "m"= 2.763
R= 2350.59

Depth of Footing = 0.35

Net BM =M1-(R*0.35/3) Kg-m
Net BM = 8632.58

6b,cal=Net BM/Z "Kg/cm^2
51.23

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

(6o,cal/60)+(6b,cal/85)= **1.3275**
OK

CHECK OF COLUMN BY LIMIT STATE METHOD
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	873 KN		976 KN
Factored load			
Pu= 1.5P	1309 KN	Pu= 1.2 P	1171 KN
BM	----		86.33 KNm
Factored BM	----	Bmu=1.2M	103.59 KNm
Pu / fck D^2 =	0.2095		0.1874
Mu / fck D^2	----		0.03
P / fck (Chart 56)			Negligible

Mini concrete area= 13094 / 10.17 1288 cm^2 < 1963 O K

Provide **9 X 16** tor main bars

Confining Reinforcement

COLUMN 500 dia

Ash= 0.09xSxDkx(fck/fy)x(Ag/Ak-1)
stp dia 10
S= spacing of links
Fck = 25 Mpa
Fy= 415
Ag = 500 ^2
Ak = 440 ^2
Dk= 440

S= 113.01 say= 100.00 mm

Provide10 tor stps @ **100** c/cnear joint for the height of 500 mm

Design of Brace M250 conc DES IN M-250 B= 250 D= 350

Span = **4.342** m A = **0.0875** Sq m

M = ***7140*** Kgm M = ***7140*** Kgm

$$R = \frac{7140 + 7140}{4.342} = 3289 \text{ Kg}$$

Self wt = 218.75 Kg m

$$SF = 3288.9 + \frac{218.75 \times 4.342}{2} = 3763.832 \text{ Kg}$$

$$\text{Net BM} = 7140.3 + \frac{218.8 \times 4.342^2}{12} - \frac{3764 \times 0.5}{3}$$

$$= 7140.3 + 343.674 - 627.3 = 6856.63 \text{ Kg m}$$

$$As = \frac{6{,}856.63 \times 100}{2300 \times 24} = 12.42 \text{ Sq cm}$$

Provide at top and bottom 2 x 20 tor str

Extra at top and bot 2 x 20 tor near support upto 1085 OK As pro 12.566

$$\text{Tau v} = \frac{3763.83}{0.25 \times 31.5} = 4.78 \text{ Kg/cm2}$$

$$\frac{100\,As}{bd} = 1.596 \qquad \text{Tau c} = 4.538 \text{ Kg/sqcm}$$

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

Footing for column as per Pg 410 of Jain & Jaikrishna

FOOTING FOR COLUMNS 4 Nos ***FOR SBC*** ***10 (T/m^2)***

GRADE OF CON	CAST IN M-250, DESIGN IN M-250					**500**	dia		
Max axial load =	**87294**	kg							
SBC =	10	(T/m^2)	Col size	b =	0.354 m		d =	0.354	9.17
Axial Load=	87294	Kg	A fact=	1.12					1.5
Area req=	9.776928	m^2	Size req =	3.1268	beq	0.709359217			
Provide sq foot of size =	**3.2**	m, A pro=		10.24	> A req OK Else		a=	3.2	
Design pressure =	8524.8047	Kg/m^2 < SBC OK					c=	3.2	
SF=	38824.64	Kg	Offset =	1.423223	m				
BM=	27628.07	Kg-m	Cover	5	cm at ends				
d req =	65.17146	cm	D=	70.67146		DIA		10	
Provide footing of depth	**75**	cm red to		**20**	cm at ends				
As =	19.48451	cm^2	As req	24.8084	x		**10**	tor	
So =	29.98852	cm	As req	9.545643	x		**10**	tor	
Asmin=	20.47279	cm^2	As req	26.06671	x		**10**	tor	
Provide bothways at bo	26.066708	**Say 28 x**		**10**	tor				
Provide	**3200 x**		**3200 x**		**750 /200 thk footing**				

Footing for column as per Pg 410 of Jain & Jaikrishna

FOOTING FOR COLUMNS 4 Nos			*FOR SBC*	*15*	*(T/m^2)*				
GRADE OF CON		CAST IN M-250, DESIGN IN M-250					**500 dia**		
Max axial load =		**87294**	kg						
SBC =	15	(T/m^2)	Col size	b =	0.354	m	d =	0.354	9.17
Axial Load=	87294	Kg	A fact=	1.1					1.5
Area req=	6.40156	m^2	Size req =	2.5301	beq	0.634359217			
Provide sq foot of size =		**2.6**	m, A pro=	6.76	> A req OK Else		a=	2.6	
Design pressure =		12913.314	Kg/m^2 < SBC OK				c=	2.6	
SF=	37711.79	Kg	Offset =	1.123223	m				
BM=	21179.38	Kg-m	Cover	5	cm at ends				
d req =	60.33987	cm	D=	65.83987		**DIA**		**10**	
Provide footing of depth		**70**	cm red to	**20**	cm at ends				
As =	16.11273	cm^2	As req	20.51532	x		**10** tor		
So =	31.42256	cm	As req	10.00211	x		**10** tor		
Asmin=	15.98673	cm^2	As req	20.35488	x		**10** tor		
Provide bothways at bo		20.51532	**Say 22 x**	**10**	tor				
Provide	**2600**	**x**	**2600**	**x**		**700 /200 thk footing**			

Footing for column as per Pg 410 of Jain & Jaikrishna

FOOTING FOR COLUMNS 4 Nos			*FOR SBC*	*20*	*(T/m^2)*				
GRADE OF CON		CAST IN M-250, DESIGN IN M-250					**500 dia**		
Max axial load =		**87294**	kg						
SBC =	20	(T/m^2)	Col size	b =	0.354	m	d =	0.354	9.17
Axial Load=	87294	Kg	A fact=	1.1					1.5
Area req=	4.80117	m^2	Size req =	2.1912	beq	0.584359217			
Provide sq foot of size =		**2.2**	m, A pro=	4.84	> A req OK Else		a=	2.2	
Design pressure =		18035.95	Kg/m^2 < SBC OK				c=	2.2	
SF=	36632.66	Kg	Offset =	0.923223	m				
BM=	16910.06	Kg-m	Cover	5	cm at ends				
d req =	56.17564	cm	D=	61.67564		**DIA**		**10**	
Provide footing of depth		**70**	cm red to	**20**	cm at ends				
As =	12.86474	cm^2	As req	16.37986	x		**10** tor		
So =	30.5234	cm	As req	9.7159	x		**10** tor		
Asmin=	13.70673	cm^2	As req	17.45191	x		**10** tor		
Provide bothways at bo		17.451905	**Say 18 x**	**10**	tor				
Provide	**2200**	**x**	**2200**	**x**		**700 /200 thk footing**			

150,000 ltrs rcc esr on 12 m stg | **FOUNDATION FOR** 7.5 T/M2

Design of Raft Foundation | SBC = 7.5 T/M^2

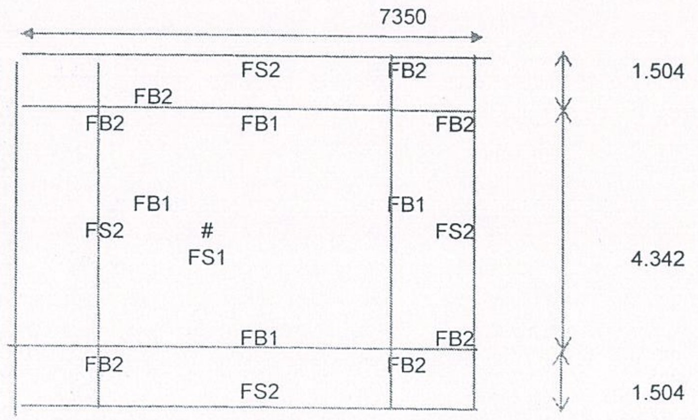
Factor 1.15

Wt of ESR = 291037 + 58139 = 349176 Kg

span = 4.342 m

Area required = $\dfrac{349176 \times 1.15}{7500}$ = 53.55 m^2

Provide Area 7350 X 7350 54.0225 m^2



P = $\dfrac{349176}{54.0225}$ = 6464 Kg/m^2

Foundation slab FS1 Panel 4.342 X 4.342 250 th

-ve BM = 0.032 X 6464 X 4.342^2 = 3900 Kgm

+ve BM = 0.024 X 6464 X 4.342^2 = 2925 Kgm

Net -ve BM = 3900 - $\dfrac{(3900 + 2925) \times 4 \times (4.342 - 0.3) \times 0.115}{4.342^2}$ = 3227 Kgm

-ve As = $\dfrac{322700}{0.87 \times 2300 \times 19.5}$ = 8.28 cm^2

+ve As = $\dfrac{292500}{0.87 \times 2300 \times 19.5}$ = 7.5 cm^2

provide at top **10 tor @** **100 c/c** bothways in square mesh at bottom, alternate
bar bent down @ **870** from support 7.86 cm^2 OK

provide at bottom **12 tor @** **200 c/c** extra near supp upto **1085** 9.584 cm^2

-ve As provided = 9.584 cm^2 **OK**

+ve As provided = 7.86 cm^2 **OK**

Foundation slab FS2 Cantilever span 1.504 - 0.15 = 1.354

$$BM = \frac{6464 \times 1.354^2}{2} = 5926 \text{ Kgm}$$

$$-ve\ As = \frac{5926 \times 100}{0.87 \times 2300 \times 34.5} = 8.59 \text{ cm}^2$$

Provide 400 reducing to 200

Provide at bottom	12 tor @	200 c/c str	i.e.9.5818	OK
	+ 10 tor @	200 c/c str		4.8744
Provide Dist-	10 x	8 tor c/c str		9.69731
				SAY

Corner Slab FS3 400 reducing to 200

continue dist 10 x 8 c/c from FS2 to form a mesh at bottom

Provide 4 x 12 tor diagonal bars at bottom at external corners only

Raft Beam FB1 300 X 800 span= 4.342

$$Load = \frac{6464 \times 4.342^2}{4} = 30467 \text{ Kg} \quad \text{Triangular}$$

udl = 1.504 X 6464 = 9722 Kg/m

$$SF = \frac{30467 + 9722 \times 4.342}{2} = 36340 \text{ Kg}$$

$$(-ve)\ BM = \frac{5}{48} \times 30467 \times 4.342 + \frac{9722 \times 4.342^2}{12} - \frac{36340 \times 0.4}{3}$$

= 13780 + 15275 - 4846 = 24209 Kgm

(+ve) BM = 13780 X 0.6 + 15275 X 0.5 = 15905.5 Kgm

$$(-ve\ As) = \frac{2420900}{0.87 \times 2300 \times 74.5} = 16.24\ cm^2$$

$$(+ve\ As) = \frac{1590550}{0.87 \times 2300 \times 74.5} = 10.67\ cm^2$$

$$As\ min = \frac{0.85}{415} \times 30 \times 74.5 = 4.58\ cm^2$$

provide at top	3 X	16 tor str				
	2 X	20 tor	extra in centre piece of length		3040	12.32
						OK
Provide at bottom	3 X	20 tor str				
	3 X	20 tor	extra near support upto	1090		18.850
						OK

$$\frac{100\ As}{b\ d} = \frac{18.85 \times 100}{30 \times 74.5} = 0.843 \qquad \text{Hence tau c} = 3.749\ Kg/cm^2$$

$$Net\ SF = 36340 - 3.749 \times 30 \times 74.5 = 27961\ Kg$$

$$10\ tor\ stps\ @ \frac{1.57079633 \times 2300 \times 74.5}{27961} \quad 9.63\ say \quad 90$$

supp to	1260	10 tor	stps @	90 c/c
1260	to centre	10 tor	stps @	200 c/c

Raft Beam FB2 300 X 800 span= 1.504

provide at top	3 X	16 tor str		
Provide at bottom	3 X	20 tor str		
	+3 X	20 tor	Str	18.850

Provide 8 tor stps @ 150 c/c

STAIRCASE : M250

Total Rise =	2.763	From GL to LDL total F	12 m
		Rise per flight =	2.763 m
span =	4342		
		As per NIT	
		1) Width of Flight =	1000 mm
		2) Flight of Rise =	2.763 m
		3) no of risers =	19 nos
		4) Rise =	145.422 mm
		5) No of treads =	18 nos
		6) Tread =	241.223

span = 4342 m

$$\text{Tread} = \frac{4342}{18} = 241.223 \text{ m}$$

WAIST SLAB : 150 THK 1000 wide span = 4342

$$\text{Load self} = \frac{375}{0.8565} = 438 \text{ Kg/m}^2$$

$$\text{Steps} = \frac{0.145422}{2} \times 2500 = 182 \text{ Kg/m}^2$$

LL = 300 Kg/m^2

Finish = 0 Kg/m^2

----

920 Kg/m^2

$$\text{SF} = \frac{920 \times 4.342}{2} = 1998 \text{ Kg}$$

$$\text{BM} = \frac{920 \times 4.342^2}{12} = 1446 \text{ Kgm}$$

d = 150 - 25 - 5= 120 mm

Mu = 1446 X 1.5 = 2169 Kgm

As = 6.26 cm2/m

So As total= 6.26 cm^2/1m

Provide **9 X** **10 tor** str at top and bottom

7.069 OK

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

CANTILEVER BEAM: 250 X 350 span 1

Cantilever span 1 m

Load Self = 219 Kg/m^2

Stair = 1998 1998 Kg/m^2

2217 Kg/m^2

SF = 2217 Kg

$$BM = \frac{2217 \times 1^2}{2} = 1108.5 \text{ Kgm}$$

Mu = 1662.75 Kgm

d = 35 - 2.5 + 1.0 = 31.5

As = 1.83 cm2

Provide at top	2x	20 tor str	12.567 sqcm
	2x	20 tor str	
At bottom	2x	20 tor str	12.567 sqcm
	2x	20 tor str	

stps - 8 tor @ 100 c/c

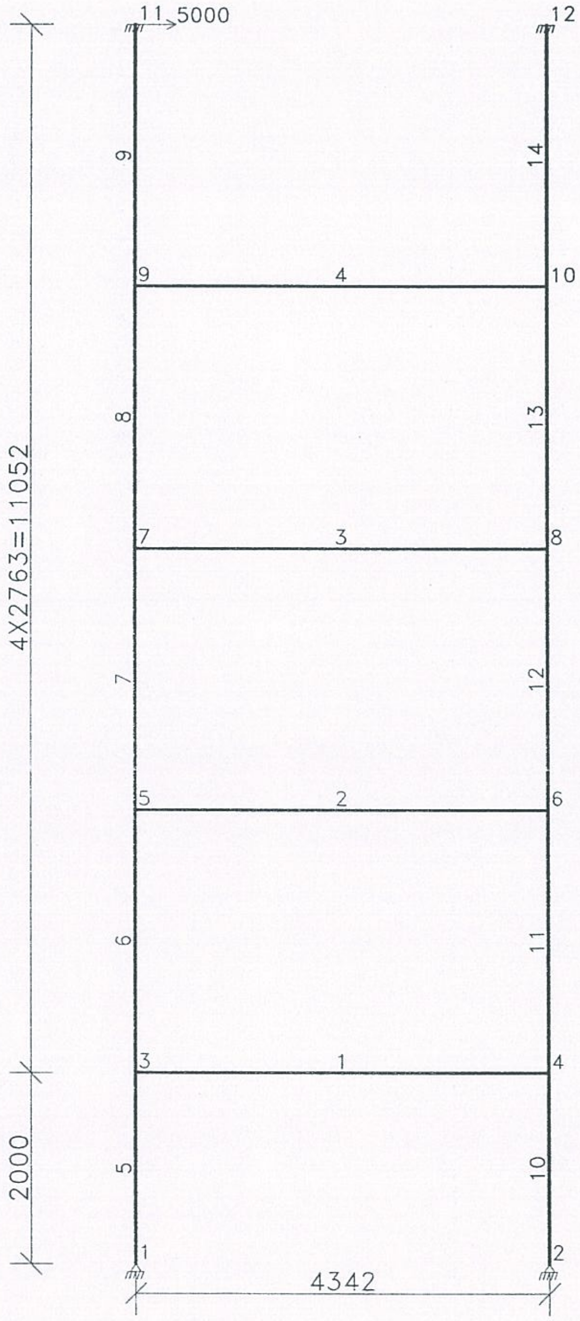
0 kl – 12 m staging MP

Load 1: seis, nodal connectivity

SCALE = 1:76	UNITS: kg m	DATE:04-06-18

X2

X1



4)

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	0.00138	0.00003	-0.0011362
4	0.00138	-0.00003	-0.0011362
5	0.00602	0.00007	-0.0017848
6	0.00602	-0.00007	-0.0017848
7	0.01167	0.00010	-0.0018581
8	0.01167	-0.00010	-0.0018581
9	0.01681	0.00011	-0.0014233
10	0.01681	-0.00011	-0.0014233
11	0.01938	0.00011	0.0000000
12	0.01938	-0.00011	0.0000000
MAX.	0.01938	0.00011	-0.0018581
NODE	11	9	7

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

Bm.	Node	Axial	V2	M3
1	3	0.000	-1564.833	-3397.252
	4	0.000	1564.833	-3397.252
2	5	0.000	-2445.930	-5310.115
	6	0.000	2445.930	-5310.115
3	7	0.000	-2532.108	-5497.206
	8	0.000	2532.108	-5497.206
4	9	0.000	-1917.820	-4163.587
	10	0.000	1917.820	-4163.587
5	1	-8460.691 T	2500.000	6857.175
	3	8460.691 T	-2500.000	-1857.175
6	3	-6895.858 T	2500.000	5254.427
	5	6895.858 T	-2500.000	1653.073
7	5	-4449.928 T	2500.000	3657.042
	7	4449.928 T	-2500.000	3250.458
8	7	-1917.820 T	2500.000	2246.748
	9	1917.820 T	-2500.000	4660.751
9	9	0.000	2500.000	-497.164
	11	0.000	-2500.000	7404.663
10	2	8460.691 C	2500.000	6857.175
	4	-8460.691 C	-2500.000	-1857.175
11	4	6895.858 C	2500.000	5254.427
	6	-6895.858 C	-2500.000	1653.073
12	6	4449.928 C	2500.000	3657.042
	8	-4449.928 C	-2500.000	3250.458
13	8	1917.820 C	2500.000	2246.748
	10	-1917.820 C	-2500.000	4660.751
14	10	0.000	2500.000	-497.164
	12	0.000	-2500.000	7404.663

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

Bm.	Node	Axial	V2	M3
MAXIMUM		8460.691 C	-2532.108	-7404.663
Beam no.		10	3	14

Verified
3/8/18

प्राध्यापक
PROFESSOR
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