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.

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

1.00 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
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email : aquades@rediffmail.com

Data :

Capacity	:	100 m^3
FSL	:	15.7 m
LDL	:	12.0
GL	:	0.00
FL	:	- 2.5
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/18023/1, 2 dt - 19/3/2018

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

100,000 ltrs rcc esr on 12 m stg

CAPACITY	100 m^3
FSL	15.7 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= 15.7-12 = 3.7 m

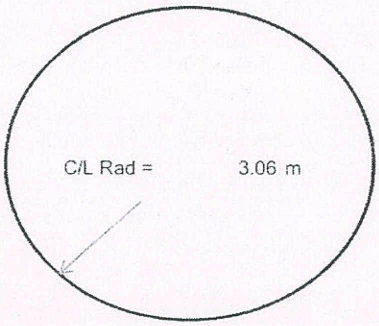
Area= 100/3.7 27.028 m^2

clear radius = 2.934 say = **2.94**

Assume wall thickness = 200 mm

Hence centre line radius = **3.06**

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

$$BM = \frac{2}{16} \times 550 \times 3.06^2 = 644 \text{ Kgm}$$

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=	6.12
dia of central shaft d=	1
wt of central shaft P=	500
p =	3.141593

$$= \frac{500 \times 6.12}{2 \times p} \times \left(1 - \frac{2 \times 1}{3 \times 6.12}\right) = 434$$

3/8/18

प्राध्यापक
PROFESSOR

Mm = mean +ve BM across diameter

$$= \frac{Mtotal}{D} = \frac{434}{6.12} = 71 \text{ Kgm}$$

For restrained edges

$$\text{-ve BM at edge} = \frac{71}{3} = 23.67 \text{ Kgm}$$

Hence BM max = 667.67 Kgm

$$As = \frac{66767}{0.87 \times 1500 \times 10} = 5.12 \text{ cm}^2$$

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

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

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 5.12 cm^2

$$\frac{100\ As}{b\ d} = \frac{5.12}{15} = 0.342$$

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 3.06^2) 5.424 m

$$\text{Hence depth required} = \frac{5.424}{39} = 0.139077 \text{ OK}$$

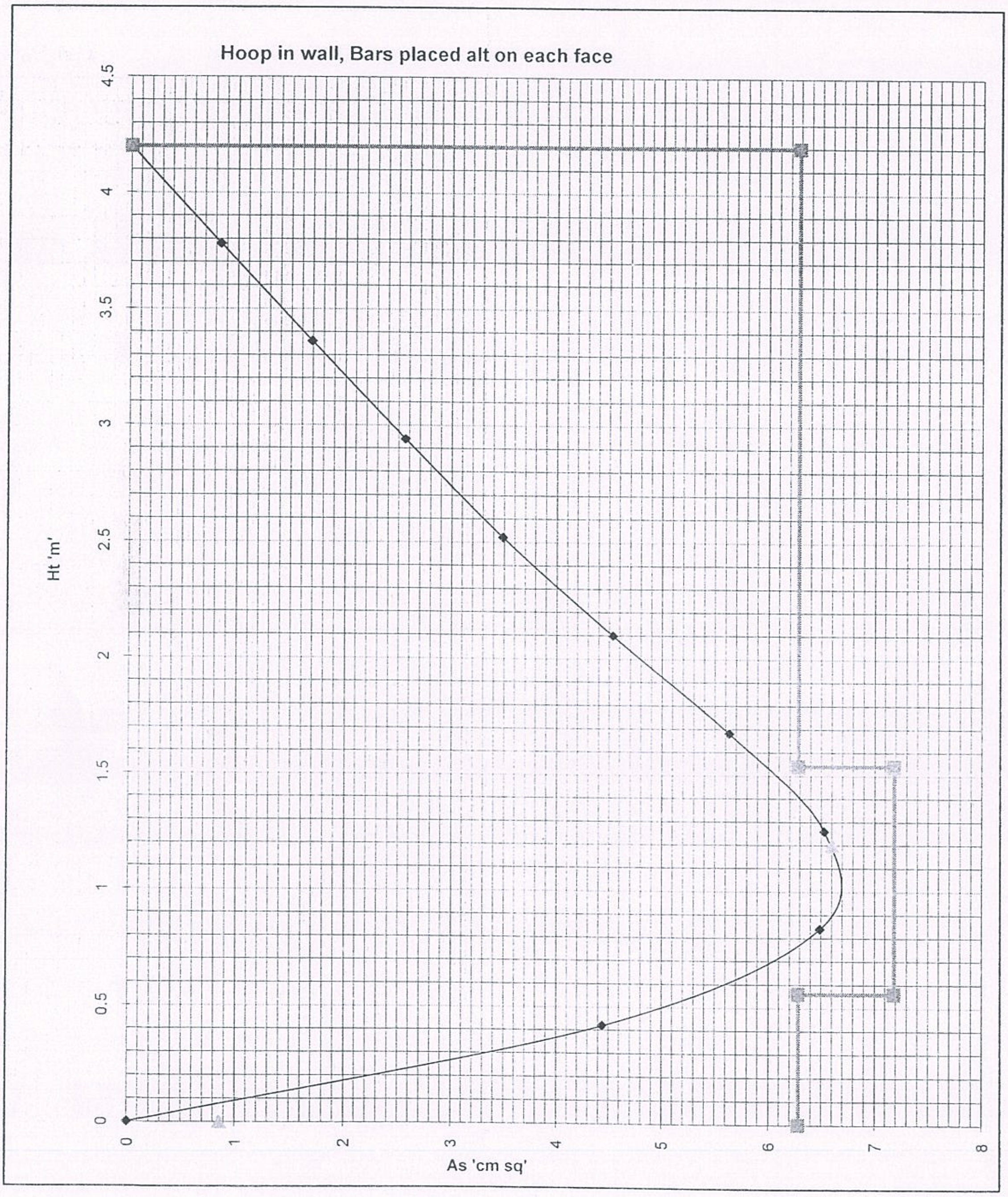
H=	4.2	D=	6.12	t=	0.2	(m-1)=	8
w =	1000						
H^2/Dt=	14.41176	fact	0.411765	fact2	2		

100,000 ltrs rcc esr on 12 m stg

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	0.0004	0.0984	0.1972	0.2990	0.4070	0.5289	0.6571	0.7616	0.7569	0.5177	
T , kg=	5	1265	2534	3843	5230	6798	8446	9788	9728	6654	0
As =	0.004	0.843	1.690	2.562	3.487	4.532	5.630	6.526	6.485	4.436	0
Ht=	**4.2**	**3.78**	**3.36**	**2.94**	**2.52**	**2.1**	**1.68**	**1.26**	**0.84**	**0.42**	0
Hta	0.002646	0.630268	1.258738	1.901885	2.579219	3.338457	4.129816	4.769657	4.741111	3.268963	



DESIGN OF WALL 200 THICK

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

Wall :
****

*C LRadius 'R' m :	3.06
*Thkof wall 'Tw' m :	0.2
*Htof Water 'h' m :	4.2
Hoop Ten 'HT' Kgm :	12348
Disp"HT*R/TwE" 'd' :	188924.4
Slope"d/h" 'Theta' :	44982
u "(3/(RTw)^2)^.25":	1.682304484
Z = "(ETw^3/12)" :	0.000666667
Mom Stiffness"2uZ" :	0.002243073
Cor Thrust"2 u^2 Z":	0.003773531
Thu Stiff "4 u^3 Z":	0.012696457

Base Slab :
****

*Thk of slab 'Ts' m:	0.26
Eff Width"6Ts" 'Be':	1.56
Thu Stif "EA/R1R2":	0.060802303
M St"EATs^2/12R1R2":	0.00034252

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'-	44982	y'-	188924.4	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:	0.002585592	-0.0038	-612.0142
Eq2=> Sum of Thu=0:	-0.00377353	0.0735	2228.9295

From Eqation 1 & 2		
Invert of above	418.0857526	21.4651
matrix	21.46511904	14.7077

0'= -208030.2
y'= 19645.517

HT in wall =	1284.020703
HT in B Slab =	3655.147522
HT in BotRingBeam =	0
HT in Gallery =	0
BM in wall =	71.25442464
BM in B Slab =	-71.2544246
BM in BotRingBeam =	0
BM in Gallery =	0

DESIGN OF WALL 200 THICK M300

Ht at Base = 1285 very small

$$\sigma_o = \frac{1285}{2000} = 0.6425 \ Kg/cm^2 \quad < 13 \ \text{OK}$$

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	560	8 tor	@ 80 c/c	8 nos	6.283 cm2
560 to	1540	8 tor	@ 70 c/c	14 nos	7.181 cm2
1540 to	4200	8 tor	@ 80 c/c	33 nos	6.283 cm2

VERTICAL STEEL-

BM = 71 Kgm

From Table 13 of IS 3370 = 0.0036735 x4200x4.2^2= 272.162268 Kgm

Design BM = 273 Kgm

$$\sigma_{bt} = \frac{273 \ X \ 6}{400} = 4.095 \ Kg/cm^2$$

< 18 Hence OK

$$As = \frac{273 \ X \ 100}{.87 \ X \ 1500 \ X \ 14.5} = 1.443 \ cm^2$$

Mini As = 0.313 x 10 = 3.13 cm^2

Provide **8 tor @ 160c/c** on each face of wall

As pro 314.15 OK

Ref Apendix CC2 for crackwidth check for Hoop in wall. Wc = **-0.1069 << 0.2 mm OK**

Ref Apendix CC3 for crackwidth check for Vert BM in wall. Wc = **-0.1468 << 0.2 mm OK**

DESIGN FOR DIRECT TENSION IN WALL **APPENDIX CC1**

Tension =	97883	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) =		451.84	**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) = 0.68 M pa Less Than 3 M Pa, OK

Calculation of Crack width

Apparent Strain e1 = T / As Es

e1 =	0.000681563
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 =	-0.000246841
a cr =	144.33

Crackwidth = 3 a cr em = -0.1069

ok

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

DATA

Permissible crackwidth			0.2 mm	Ms =	0.22 kNm			
Width of the section		b	1000 mm	Mu=	0.33 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=	4.667171	
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) = 2.43E-05

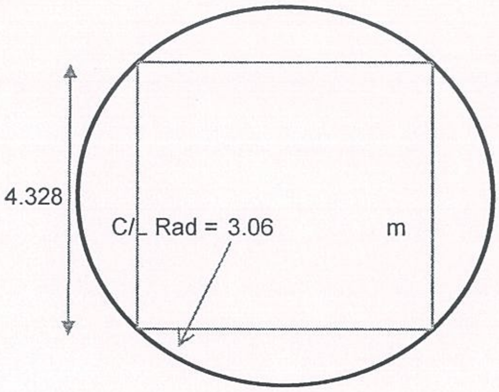
B= (1+2*(acr-c)/(h-x)) = 2.160726195

W=(A-e2)*3*acr/(B) = -0.1468 OK

LA Z = d - (x/3) =	184.52	
Steel Tensile Stress fs = Ms/(z As) =	4.741	M Pa
Conc Comp Stress = 2Ms/(z b x) =	0.069	M Pa
CHECK STRESS LEVELS		
0.45 fcu =	13.5 M Pa	OK
0.8 fy =	332 M Pa	OK

BASE SLAB 260 THICK M300

SPAN = **4.328 X** **4.328**



Loading =

DL = 650

water = 4200

plaster = 42

4892 Kg/m^2

Refer table 22, IS 456/2000, Interior Panel

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

- ve BM = 0.032 X 4892 X 4.328^2 = 2933 Kgm

+ve BM = 0.024 X 4892 X 4.328^2 = 2200 Kgm

Net -ve BM at the face of support=

$$= 2933 - \frac{(2933 + 2200) \times 4 \times (4.328 - 0.3) \times 0.15}{4.328^2} = 2271 \text{ Kgm}$$

SAY = 2933

$$6bt = \frac{2271 \times 6}{26^2} = 20.16 \text{ Kg/cm}^2$$

<20 hence OK
with steel contribution

$$-ve\ As = \frac{1.5 \times 2933000}{0.8 \times 0.87 \times 415 \times 210} = 726 \text{ mm}^2$$

$$+ve\ As = \frac{1.5 \times 2200000}{0.8 \times 0.87 \times 415 \times 210} = 545\ 3\text{m}^2$$

$$\text{Mini As} = (0.3 - \frac{16}{350}) \times 26 \times 0.8 = 5.29 \text{ cm}^2$$

Mini As = 0.313 x 13 = 406.9 mm^2

provide at bottom **10 tor @** **90 c/c** bothways in square mesh at bottom, alternate

bar bent up @ **865** from support 8.727 cm^2

provide at top **10 tor @** **180** bothways in sqmesh 4.3635 cm^2

-ve As provided = 872.7 mm^2 **OK**

+ve As provided = 872.7 mm^2 **OK**

$$\text{Hoop in base slab} = \frac{3656}{1500} = 2.44 \text{ cm}^2$$

provide 2 X **10 tor** hoop bars at top & bottom near wall (As provided=4.712cm^2) Total 4 nos

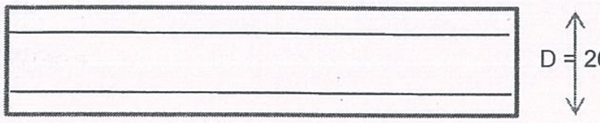
CHECK FOR CRACK WIDTH

Ref Appendix CC3, Wc = 0.1080047 << 0.2 mm OK

CHECK FOR NON CRACKING

Ast=69.816

Asb=54.5438



D = 260mm

Area of steel provided at top= (9 - 1) X 872.7 = 69.816 cm^2

Area of steel provided at bottom= (1.5 X 9 -1) X 4.3635= 54.5438 cm^2

As = (26) X 100 + 69.816 + 54.5438 = 2724.36 cm^2

$$y = \frac{2600 \times 13 + 69.816 \times 5.5 + 54.5438 \times 20.5}{2724.3598} = 12.96 \text{ cm}$$

$$I = \frac{100 \times 26^3}{12} + (2600)*(13 - 12.96)^2 + 69.816 \times 7.46^2 + 54.5438 \times 7.54^2$$

$$= 146466.67 + 4.16 + 3885.37 + 3100.902$$

$$= 153457 \text{ cm}^4$$

$$Z = 11840.8257 \text{ cm}^3$$

$$6bt = \frac{227100}{11840.8257} = 19.18 \text{ Kg/cm}^2$$ < 20 hence OK

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

DATA

Permissible crackwidth		0.2 mm	Ms =	29.33 kNm				
Width of the section	b	1000 mm	Mu=	43.995 kNm				
Overall Depth of the section	D	260 mm	Cover	45 mm				
Es		2.00E+05 Mpa						
fck		30 Mpa	Ec = 5000*sqrt(30)=	27386.13 Mpa	As req=	604.63		
Use 10 dia bars @ 90 mm c/c			*(For calculation of crackwidth use 0.5 Ec)*					
As = 872.70 sqmm per 1000mm	Fy =	415 Mpa					total steel	872.7 sqmm
acr = 62.27							Spacing	90 c/c
d=(D-c- 210.00 mm	(CHECK)						Eq Dia	10.0002 cm

r = 0.004155716 , Modulus of elasticity m = 14.60593

(x/d) = -mr+sqrt(mr(2+mr)) = 0.293 ie x= 61.53 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 0.000506682

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 0.000760022

For our 0.0005

A= (Ms/(Es*As*(d-(x/3)))*(h-x)/(d-x 1.19E-03

B= (1+2*(acr-c)/(h-x)) = 1.1740121

W=(A-e2)*3*acr/(B) = 0.10800465 OK

LA,Z = d - 189.49

Steel Tensile Stress fs 177.362 M Pa

Conc Comp Stress = 215.031 M Pa

CHECK STRESS LEVELS

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

BASE BEAM 300 X 800 M300 span= 4.328 m

$$\text{Triangular load} = \frac{4892 \times \pi \times 3.06^2}{4} = 35977 \text{ Kg}$$

$$\text{udl} = 0.3 \times 0.54 \times 2500 = 405 \text{ Kg/m}$$

$$\text{SF} = \frac{35977 + 405 \times 4.328}{2} = 18865 \text{ Kg}$$

$$\text{-veBM} = \frac{5}{48} \times 35977 \times 4.328 + \frac{405 \times 4.328^2}{12} - \frac{18865 \times 0.45}{3}$$

$$= 16220 + 633 - 2830$$

$$= 14024 \text{ Kgm}$$

$$\text{+veBM} = 0.6 \times 16220 + 0.5 \times 633 = 10049 \text{ Kgm}$$

$$\text{-veAs} = \frac{1402400}{0.87 \times 1500 \times 74.5} = 14.43 \text{ cm}^2$$

$$\text{+veAs} = \frac{1004900}{0.87 \times 1900 \times 74.5} = 8.160 \text{ cm}^2$$

$$\text{As min} = \frac{0.85}{415} \times 30 \times 74.5 = 4.578 \text{ cm}^2$$

Provide at bottom	2 X 16 tor str				
	2 X 20 tor	extra in centre piece of length	3030	OK	10.30 Sqcm
Provide at top	2 X 20 tor str				
	3 X 20 tor	extra near supp upto	0	OK	15.71 Sqcm

$$\text{tau v} = \frac{18865}{30 \times 74.5} = 8.44 \text{ Kg/cm}^2$$

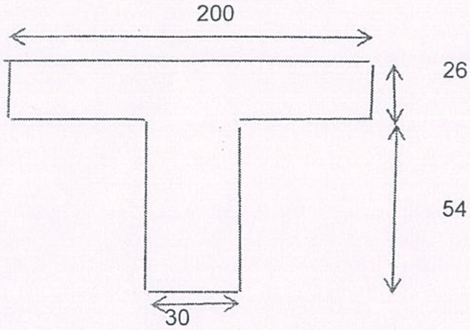
$$\frac{100 \text{ As}}{\text{b d}} = \frac{15.708 \times 100}{30 \times 74.5} = 0.703$$ Hence tau c = 3.51 Kg/cm^2

$$\text{Net SF} = 18865 - 3.506 \times 30 \times 74.5 = 11030 \text{ kg}$$

$$\text{8 tor stps @} \; \frac{1.005 \times 1750 \times 74.5}{11030} = 11.88 \text{ say } 110$$

supp to	1320	8 tor	stps @	110 c/c
1320	to centre	8 tor	stps @	200 c/c

NON CRACKING CHECK



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

$$= \frac{0.7 \times 4.328}{6} + 0.3 + 6 \times 0.26 = 2.365 \quad m$$

$$bf\ available = \frac{2.365}{2} + 3.06 \times (1 - .707) = 2.07908 \ m$$

say 200 cm

$$A = 5200 + 1620 = 6820 \ cm^2$$

$$Y = \frac{5200 \times 13 + 1620 \times 53}{6820} = 22.502 \quad cm$$

$$I = \frac{200}{12} \times 26^3 + 5200 \times 9.5^2 + \frac{30}{12} \times 54^3 + 1620 \times 30.5^2$$

$$= 292933 + 469498 + 393660 + 2E+06 = 2662898 \ cm^4$$

$$Z = \frac{2662898.32}{22.502} = 118340.5 \ cm^3$$

$$6bt = \frac{1004900}{118340.51} = 8.5$$ OK

DESIGN OF GALLERY 120 THICK M300 Span = 1 m cantilever

DL = 300
LL = 300

Total = 600 Kg/m^2

BM = 300 m Kgm

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

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

Check for wall as beam 200 wide X 4610 deep
It is tied at top by roof slab and at bottom by base slab

Roof Slab = 17254 Kg
Self Wt = 40376 Kg
Gallery = 13798 Kg

TOTAL 71428 Kg

$$WR = \frac{71428}{2 \times 3.1415} = 11369 \text{ Kg}$$

$$WR2\ (2a) = \frac{11369 \times 3.1416 \times 3.06}{2} \quad 54647$$

-ve BM = 0.137 X 54647 = 7487 Kgm
+ve BM = 0.07 X 54647 = 3826 Kgm
Torsion = 0.021 X 54647 = 1148

$$SF \text{ at support} = 11369 \times \frac{3.141593}{4} = 8930 \text{ Kg}$$

$$SF \text{ at contra} = 11369 \times \left(\frac{3.141593}{4} - 0.336\right) = 5110 \text{ Kg}$$

$$6bt = \frac{748700 \times 6}{20 \times 461\text{^}2} = 1.0568838 \text{ Kg/cm}^2$$
< 18 hence OK

Clause 28, IS 456/1978, for deep beam

$$\frac{L}{D} = \frac{2 \times 3.142 \times 3.06}{4} \times \frac{1}{4.61} = \frac{4.81}{4.61} = 1.043 < 2$$

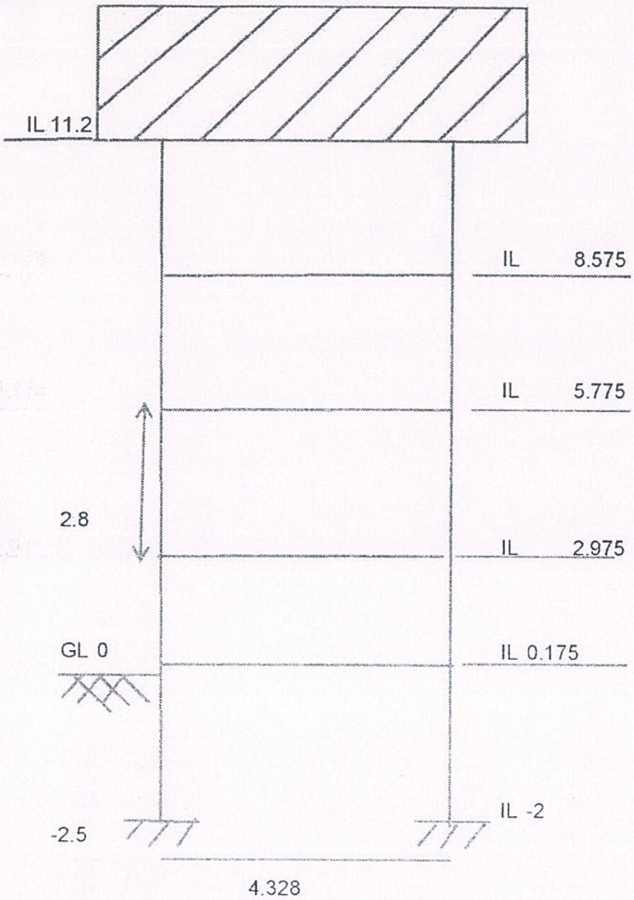
Hence Z = 0.2 X (4.81 + 2 X 4.6) = 2.806 m

$$As = \frac{748700}{1500 \times 280.6} = 1.78 \text{ cm}^2 \quad \text{very small}$$

$$\text{Net SF} = 5110 + \frac{1.6 \times 1148}{0.2} = 14294 \text{ Kg}$$

$$\text{Tau v} = \frac{14294}{20 \times 456} = 1.57 \text{ Kg/cm}^2$$
very small
Hence O K

Staging - consider 450 dia columns brace 250 350



COLUMN 450 4 Nos

$$A = \frac{3.1416 \times 45^2}{4} = 1590 \text{ cm}^2$$

$$I = \frac{3.1416 \times 45^4}{4} = 3220623 \text{ cm}^4$$

Brace 250 X 350 span= 4.328
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= 0.02352 m

WEIGHT OF CONTAINER

Roof Slab	=	3.1416 X 3.16^2 X 550 =	17254
wall	=	2 X 3.1416 X 3.06 X 4.2 X2500 X 0.2 =	40376
Base Slab	=	3.1416 X 3.16^2 X 650 =	20391
Base Beam	=	0.162 X 4.328 X 2500 X 4 =	7011
Gallery	=	3.1416 X (4.16^2 - 3.16^2) X 600 =	13798
water	=	3.14159 X 2.94^2 X 4200 =	114050
plaster	=	(3.1416 X 2.95^2+2 X3.14X3 X 4.2) X 2080X0.02	4376

		TOTAL =	**217256** Kg

WEIGHT OF STAGING

column dia = 400 brace= 250 X 350 span= 4.328
staging ht= 12

Column	=	0.1590 X 4 X 13.7 X 2500 =	21789
Brace	=	4 X 4 X 0.0875 X 3.928 X 2500 =	13748

		TOTAL =	**35537** Kg

ADD FOR STAIR

MB1 = 8.0000 X 2243 = 17944

Total Stg wt = 35537 + 17944 = **53481** Kg

Deduct Live Load

Roof Slab	=	3.1416 X 3.16^2 X 75 =	2353
Gallery	=	3.1416 X (4.16^2 - 3.16^2) X 300 =	6899

		TOTAL =	**9252** Kg

Net Wt of Container= 217256 - 9252 = **208004** Kg

Deduct LL on Stair = $\frac{300}{935}$ X 53481 = 17160 Kg

Net Staging Load = 53481 - 17160 = **36321** Kg

SEISMIC ANALYSIS

PARAMETER	STAGING
	12 M
DEFLECTION	0.02352

A) TANK FULL

1. W= (208004+(36321/3)) Kg
   = 220111

2. d AT TOP= (220111/10) X 0.02352 m
   = 0.5177

3. PERIOD T= 2 X 3.14 X SQRT(d/9.81) sec
   = 1.4432

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

4. Sa/g= **1.67/T =** 1.1572

5. ah= (Z/2) (I/R) (Sa/g) z= 0.16
   = 0.046288 I= 1.5
   = 0.046288 R= 3

6. SEISMIC FORCE = W X ah Kg
   = **10188**

WT OF WATER 114050

B) TANK EMPTY

1. W= (220111-114050) Kg
   = 106061

2. d AT TOP= (106062/220111)*0.5178m
   = 0.2495

3. PERIOD T= 2 X 3.14 X SQRT(d/9.81) sec
   = 1.0019

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

4. Sa/g= **1.67/T =** 1.6668

5. ah= (Z/2) (I/R) (Sa/g) z= 0.16
   = 0.066672 I= 1.5
   = 0.066672 R= 3

6. SEISMIC FORCE = W X ah Kg
   = **7071**

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

Design wind speed at any height Z m/sec

Vz = Vb X K1 X K2 X K3

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 = 14.575) = 0.965

K3 = 1.0 (Topography factor)

Hence Vz = 37.63 m/sec

Design wind pressure Pz = 0.6×37.631^2 = 850 N/m^2

= 85.0 Kg/m^2

From appendix D, Reynold's number

$$R = \frac{D\,Vd}{Y} = \frac{6.32 \times 37.63}{1.46 \times 10^{-5}} = 16{,}289{,}627$$

and E/D = 0.001

$$\text{From fig (5),} \quad \frac{Cf}{1 + 2\,E/D} = 0.81 \qquad \text{Hence } Cf = 0.81162$$

Hence wind on container = Cf Ae pd

= 0.81162 X 6.32 X 5.15 X 85.0

= 2245 Kg

Column :

$$R = \frac{D\,Vd}{Y} = \frac{0.45 \times 37.63}{1.46 \times 10^{-5}} = 1{,}159{,}863$$

and E/D = 0.001

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

Hence wind on column = Cf Ae pd

= 0.9018 X 11.2 X 85 X 4 X 0.45

= 1545 Kg

$$\text{Hence equivalent wind at top} = \frac{1545}{2} = 772 \text{ Kg}$$

$$\textbf{Braces :} = \frac{3 \times 1.36 \times 0.35 \times 3.878 \times 2 \times 85.0}{2}$$

= 472 Kg

Total wind force at top = 2245 + 772 + 472

= 3489 Kg

Design of column-

Critical load is	seismic		
	10188		
a) HORZ DESIGN	seismic		
FORCE (Kg)	10188		

ALLOWABLE STRESSES CAN BE INCREASED BY 1.333

b) Mult Factor	10188/1000		
	1.0188		

REFER APPENDIX -I

Max BM in Col	1.0188 X		
@ Bot Kg-m	5080		
M1=	5176		
@ Top Kg-m	732		
M2=	746		
	1.0188 X		
Additional Axial	5470		
Load in Col Kg	5573		
Vert Seismic	Seismic force/ (2 X 4)		
Force/Col ' Kg' =	1274		
Max BM in Brace	1.0188 X		
Kg - m	-6530		
=	-6653		
Max Story Drift =	0.00664 X	1.018849797 =	0.006765163 m
Panel =	2.8	P=	270737 Kg
		Base Shear=	10188

STABILITY INDEX = $\dfrac{0.00677 \times 270737}{2.80000 \times 10188}$ **=0.064203346** <<< 1

DESIGN OF COLUMN : 450 DIA SIZE, 4 Nos, Design in M250 Conc, Cast in M-250

DIA OF COL 'd' mm = **450** Area of Column 'Ac' ı Area of Column 'Ac' cm^2= **1590**

PARAMETER STAGING

12 m

a) WT OF CONTAINER 217256

b) NET WT OF CONT 208004

c) WT OF STAGING 53481

d) AXIAL LOAD/COL'Kg' (Wt Of Cont + Wt Of Staging)/4
= 67684 Non seismic

e) Ac Required ' cm^2' = Axial Load/74.7
= 906.08

f) As Required 'cm^2' = Ac x 0.008
= 7.249 0.456 OK

g)As Provided
Dia of Bar (TOR) 12
No of Bars 7
Area 'cm^2' "Asp" 7.92

Provide 8 TOR links at 150 c/c

h)Area of Compo Sec
Ag=Ac+(1.5x11-1)xAsp 1713.14

I=3.14x(d^4)/64+.5x15.5Aspx(d/2-4.8)^2
= 240606.66
Z=I x 2/d = 10693.63

INTERACTION CHECK
Axial Load seismic

'Ph' = Net wt of ESR + Additional Axial Load due to Horz Force+Vert Seismic Force "Kg"
= 72218

6o,cal= Ph/Ag " Kg/cm^2"
42.16

R(Shear in col)=(M1+M2)/(LA) LA "m"= 2.8
R= 2114.84
Depth of Footing = 0.35
Net BM =M1-(R*0.35/3) Kg-m
Net BM = 4929.03

6b,cal=Net BM/Z "Kg/cm^2
46.09

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

(6o,cal/60)+(6b,cal/85)= **1.2449**
OK

CHECK OF COLUMN BY LIMIT STATE METHOD
REFER SP :16/1980 DESIGN AIDS TO IS 456/2000

D = 450 d'/D = 46/450 0.102222

fck = 25 fy = 415

condition	Non seismic		Seismic
P	677 KN		722 KN
Factored load Pu= 1.5P	1015 KN	Pu= 1.2 P	867 KN
BM	----		49.29 KNm
Factored BM	----	Bmu=1.2M	59.15 KNm
Pu / fck D^2 =	0.2005		0.1712
Mu / fck D^2	----		0.03
P / fck (Chart 56)			Negligible

Mini concrete area= 10153 / 10.17 998 cm^2 < 1590 O K

Provide **7 X 12** tor main bars

Confining Reinforcement

COLUMN 450 dia

Ash= 0.09xSxDkx(fck/fy)x(Ag/Ak-1)
stp dia 10
S= spacing of links
Fck = 25 Mpa
Fy= 415
Ag = 450 ^2
Ak = 390 ^2
Dk= 390

S= 112.10 say= 100.00 mm

Provide10 tor stps @ **100** c/cnear joint for the height of 450 mm

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

Span = **4.328** m A = **0.0875** Sq m

M = **6653** Kgm M = **6653** Kgm

$$R = \frac{6653 + 6653}{4.328} = 3074 \text{ Kg}$$

Self wt = 218.75 Kg m

$$SF = 3074.4 + \frac{218.8 \times 4.328}{2} = 3547.82 \text{ Kg}$$

$$\text{Net BM} = 6653.1 + \frac{218.8 \times 4.328^2}{12} - \frac{3548 \times 0.45}{3}$$

$$= 6653.1 + 341.46 - 532.2 = 6462.38 \text{ Kg m}$$

$$As = \frac{6{,}462.38 \times 100}{2300 \times 24} = 11.71 \text{ Sq cm}$$

Provide at top and bottom 3 x 16 tor str

Extra at top and bot 2 x 20 tor near support upto 0 OK As pro 12.315

$$\text{Tau v} = \frac{3547.82}{0.25 \times 31.5} = 4.51 \text{ Kg/cm2}$$

$$\frac{100\,As}{bd} = 1.564 \qquad \text{Tau c} = 4.526 \text{ Kg/sqcm}$$

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				**450** dia		
Max axial load =	**67684** kg						
SBC =	10 (T/m^2)	Col size	b =	0.318 m	d =	0.318	9.17
Axial Load=	67684.25 Kg	A fact=	1.12				1.5
Area req=	7.580636 m^2	Size req =	2.7533 beq	0.628423295			
Provide sq foot of size =	**2.8** m, A pro=		7.84 > A req OK Else		a=	2.8	
Design pressure =	8633.1952 Kg/m^2 < SBC OK				c=	2.8	
SF=	29996.23 Kg	Offset =	1.240901 m				
BM=	18611.18 Kg-m	Cover	5 cm at ends				
d req =	56.8298 cm	D=	62.3298	DIA	**10**		
Provide footing of depth	**65** cm red to		**20** cm at ends				
As =	15.36907 cm^2	As req	19.56846 x		**10** tor		
So =	27.12995 cm	As req	8.635731 x		**10** tor		
Asmin=	16.07459 cm^2	As req	20.46676 x		**10** tor		
Provide bothways at bo	20.466761	**Say 21 x**	**10** tor				
Provide	**2800 x**	**2800 x**		**650 /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			**450 dia**		
Max axial load =		**67684**	kg					
SBC =	15	(T/m^2)	Col size	b =	0.318 m	d =	0.318	9.17
Axial Load=	67684.25	Kg	A fact=	1.1				1.5
Area req=	4.963512	m^2	Size req =	2.2279 beq	0.565923295			
Provide sq foot of size =		**2.3**	m, A pro=	5.29	> A req OK Else	a=	2.3	
Design pressure =		12794.754	Kg/m^2 < SBC OK			c=	2.3	
SF=	29160.17	Kg	Offset =	0.990901	m			
BM=	14447.42	Kg-m	Cover	5	cm at ends			
d req =	52.76327	cm	D=	58.26327	**DIA**		**10**	
Provide footing of depth		**65**	cm red to	**20**	cm at ends			
As =	11.93065	cm^2	As req	15.19054 x		**10** tor		
So =	26.37378	cm	As req	8.395034 x		**10** tor		
Asmin=	13.37459	cm^2	As req	17.02902 x		**10** tor		
Provide bothways at bo	17.029022		**Say 18 x**	**10** tor				
Provide	**2300 x**		**2300 x**		**650 /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			**450 dia**		
Max axial load =		**67684**	kg					
SBC =	20	(T/m^2)	Col size	b =	0.318 m	d =	0.318	9.17
Axial Load=	67684.25	Kg	A fact=	1.1				1.5
Area req=	3.722634	m^2	Size req =	1.9294 beq	0.528423295			
Provide sq foot of size =		**2**	m, A pro=	4	> A req OK Else	a=	2	
Design pressure =		16921.063	Kg/m^2 < SBC OK			c=	2	
SF=	28457.88	Kg	Offset =	0.840901	m			
BM=	11965.13	Kg-m	Cover	5	cm at ends			
d req =	49.69161	cm	D=	55.19161	**DIA**		**10**	
Provide footing of depth		**60**	cm red to	**20**	cm at ends			
As =	10.80421	cm^2	As req	13.75632 x		**10** tor		
So =	28.14407	cm	As req	8.958535 x		**10** tor		
Asmin=	11.05913	cm^2	As req	14.08089 x		**10** tor		
Provide bothways at bo	14.080895		**Say 15 x**	**10** tor				
Provide	**2000 x**		**2000 x**		**600 /200 thk footing**			

STAIRCASE : M250

Total Rise =	2.8	From GL to LDL total F	12	m
		Rise per flight =	2.8	m
span =	4328			
		As per NIT		
		1) Width of Flight =	1000	mm
		2) Flight of Rise =	2.8	m
		3) no of risers =	18	nos
		4) Rise =	155.556	mm
		5) No of treads =	17	nos
		6) Tread =	254.589	
		span =	4328	m

Tread = $\dfrac{4328}{17}$ = 254.589 m

WAIST SLAB : 150 THK 1000 wide span = 4328

Load self = $\dfrac{375}{0.8534}$ = 440 Kg/m^2

Steps = $\dfrac{0.155556}{2}$ X 2500 = 195 Kg/m^2

LL = 300 Kg/m^2

Finish = 0 Kg/m^2

----

935 Kg/m^2

SF = $\dfrac{935 \times 4.328}{2}$ = 2024 Kg

BM = $\dfrac{935 \times 4.328^2}{12}$ = 1460 Kgm

d = 150 - 25 - 5= 120 mm

Mu = 1460 X 1.5 = 2190 Kgm

As = 6.32 cm2/m

So As total= 6.32 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 =	2024	2024	Kg/m^2

		2243	Kg/m^2

SF = 2243 Kg

$$BM = \frac{2243 \times 1^2}{2} = 1121.5 \text{ Kgm}$$

Mu = 1682.25 Kgm

d = 35 - 2.5 + 1.0 = 31.5

As = 1.85 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	

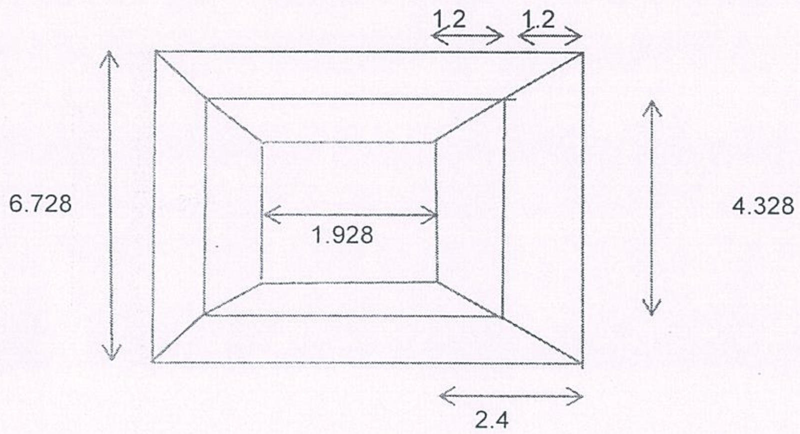
stps - 8 tor @ 100 c/c

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

Wt of ESR = 217256 + 35537 = 252793 Kg

span = 4.328 m

$$\text{Area reqd} = \frac{252793 \times 1.1}{7500} = 37.08\ m^2$$



Provide strip raft as shown in fig

Width of strip = 2.4 m

$A_{provided}$ = 6.728^2 - 1.928^2 = 41.5488 **> A reqd** OK

Design of Raft Slab Balance Cantilever = 1.2 - 0.15 = 1.05 m

$$p = \frac{252793}{41.5488} = 6085\ Kg/m^2$$

SF = 6085 X 1.05 = 6389.25 Kg

BM = 6389.25 X 0.525 = 3355 Kgm

Dreqd = sqrt(3355/11.14) = 19.13 cm

Provide 300 depth reducing to 200 at ends

$$As = \frac{335500}{0.87 \times 2300 \times 25.5} = 6.576\ cm^2$$

As min = 0.0012 X 25.5 X 100 = $3.06\ cm^2$

Provide at botttom 10 tor @ 110 c/c str bars OK As pro 7.139

Provide distribution - 6 X 10 tor @ on each side

Provide 4 X 12 tor diagonal bars at bottom corners

Raft Beam 300 X 800 span 4.328 m

Load UDL = 6085 X 1.2 X 2 = 14604 Kg/m

$$SF = \frac{14604 \times 4.328}{2} = 31604 \text{ Kg}$$

$$\text{-ve BM} = \frac{14604 \times 4.328^2}{12} - \frac{31604 \times 0.45}{3}$$

= 22797 - 4740.6 = 18057 Kgm

$$\text{-ve AS} = \frac{1805700}{0.87 \times 2300 \times 74.5} = 12.12 \text{ cm}^2$$

+ve BM = 0.5X 22797 = 11398.5 Kgm

$$\text{+ve AS} = \frac{1139850}{0.87 \times 2300 \times 74.5} = 7.65 \text{ cm}^2$$

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

Provide at TOP 3 X 16 tor str
2 X 16 tor extra in centre piece of length 3030

Provide at BOTTOM 3 X 16 tor str
2 X 20 tor extra near supp upto 0

$$\frac{100 \text{ As}}{\text{b d}} = \frac{12.32 \times 100}{30 \times \times 74.5} = 0.551$$ Hence tau c = 3.20 Kg/cm^2

Net SF = 31604 - 3.2025 X 30 X 74.5 = 24447 kg

$$\text{10 tor stps @} \quad \frac{1.5708 \times 2300 \times \times 74.5}{24447} = 11.01 \text{ say } 110$$

supp to 1100 10 tor stps @ 110 c/c
1100 to centre 10 tor stps @ 200 c/c

Venkat 8/3/18
N. Dinkulu

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