MADHYA PRADESH JAL NIGAM MARYADIT

(A Govt. of Madhya Pradesh Undertaking)
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Ph: 0755-2579034, Fax: 0755-2579873
CIN. No. U41000MP2012SGC028798

No. 4075 **/03/O&M/MPJNM/2018** **Bhopal, Dated** 7 / 8**/2018**

To,

Engineer-in-Chief
Public Health Engineering Department
Jal Bhawan, Banganga
Bhopal.

Sub:- Structural design and drawing of RCC ESRs.

The following drawings and design have been vetted by MANIT, Bhopal and are being sent along with the letter of MANIT, Bhopal for necessary action :-

1. 100 KL capacity -12 m staging considering S.B.C. 7.5, 10, 15 & 20 T/m^2 And seismic zone-3
2. 125 KL capacity -12 m staging considering S.B.C. 7.5, 10, 15 & 20 T/m^2 And seismic zone-3
3. 150 KL capacity -12 m staging considering S.B.C. 7.5, 10, 15 & 20 T/m^2 And seismic zone-3
4. 175 KL capacity -12 m staging considering S.B.C. 7.5, 10, 15 & 20 T/m^2 And seismic zone-3

SE(M) 13/8

E-in-C 10/8

Encl:- As above.

(N.P. Malvia)
Project Director

R.No - 3011/SE/M 14-8-2018

C- 7553 13-8-18

EE(M-2) A 13/8/18

AE(M) Pl. check & Discuss. 16/8

Maulana Azad National Institute of Technology, Bhopal

(An Institute of National Importance)

DEPARTMENT OF CIVIL ENGINEERING

Tel: (0755) 4051000, 4052000
5206006, Fax: (0755) 2670562
Civil Dept. 4051200, 4051201, 4051204, 4051237

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.

प्राध्यापक
PROFESSOR
सिविल अभियांत्रिकी विभाग
Department of Civil Engineering
मौलाना आजाद राष्ट्रीय प्रौद्योगिकी संस्थान,
Maulana Azad National Institute of Technology
भोपाल (म. प्र.)/BHOPAL-462007 (M.P.)

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- 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.

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3/8/18

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भोपाल (म. प्र.)/BHOPAL-462007 (M.P.)

1.25 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	:	125 m^3
FSL	:	16.61 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/18023/4, 5, 6 dt - 31/5/2018

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

125,000 ltrs rcc esr on 12 m stg

CAPACITY	125	m^3
FSL	16.61	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= 16.61-12 = 4.61 m

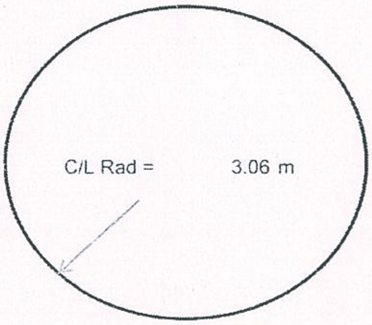
Area= 125/4.61 27.115 m^2

clear radius = 2.938 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= $\dfrac{2}{16}$ X 550 X 3.06^2 = 644 Kgm

Wt of central shaft = 500 Kg assumed

M_{total} = Total + Ve BM across diameter

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

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

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

Verified
3/8/18
N. Dinkular

प्राध्यापक
PROFESSOR
(M.P.)

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

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

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

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

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 5.12 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

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

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 :	5.11
Hoop Ten 'HT' Kgm :	15023.4
Disp"HT*R/TwE" 'd' :	229858.02
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:	0.28
Eff Width"6Ts" 'Be':	1.68
Thu Stif "EA/R1R2":	0.072512024
M St"EATs^2/12R1R2":	0.000473745

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'-	229858	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:	0.002716818	-0.0038	-766.4785
Eq2=> Sum of Thu=0:	-0.00377353	0.08521	2748.6414

From Eqation 1 & 2		
Invert of above	392.2024229	17.369
matrix	17.36902313	12.5051

0'= -252873.5
y'= 21059.117

HT in wall =	1376.412882
HT in B Slab =	4672.739944
HT in BotRingBeam =	0
HT in Gallery =	0
BM in wall =	119.7976178
BM in B Slab =	-119.797618
BM in BotRingBeam =	0
BM in Gallery =	0

DESIGN OF WALL 200 THICK M300

Ht at Base = 1377 very small

$$6o = \frac{1377}{2000} = 0.6885 \text{ Kg/cm}^2 \quad < 13 \text{ OK}$$

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	420	8 tor	@ 70 c/c	7 nos	7.181	cm2
420 to	1440	8 tor	@ 60 c/c	17 nos	8.378	cm2
1440 to	5110	8 tor	@ 70 c/c	52 nos	7.181	cm2

VERTICAL STEEL-

BM = 120 Kgm

Design BM = 120 Kgm

$$6bt = \frac{120 \times 6}{400} = 1.8 \text{ Kg/cm}^2$$

< 18 Hence OK

$$As = \frac{120 \times 100}{.87 \times 1500 \times 14.5} = 0.635 \text{ cm}^2$$

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 = -0.006 << 0.2 mm OK

Ref Apendix CC3 for crackwidth check for Vert BM in wall. Wc = -0.1455 << 0.2 mm OK

DESIGN FOR DIRECT TENSION IN WALL		**APPENDIX CC1**			
Tension =	131348	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) =		606.32	**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) = 0.85 M pa — Less Than 3 M Pa, OK

Calculation of Crack width

Apparent Strain e1 = T / As Es	
e1 =	0.000914577
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 =	-1.38268E-05
a cr =	144.33

Crackwidth = 3 a cr em = -0.006

ok

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

DATA

Permissible crackwidth		0.2 mm	Ms =	0.31 kNm			
Width of the section	b	1000 mm	Mu=	0.465 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=	6.577356
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

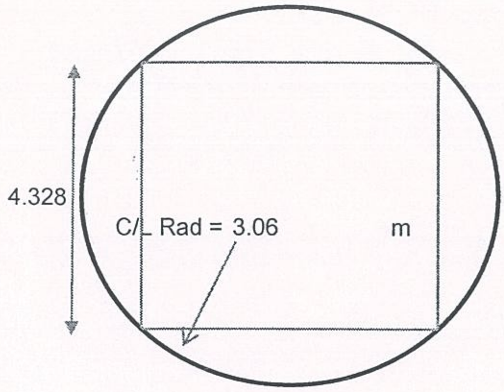
For our case e2 = 0.0011

A= (Ms/(Es*As*(d-(x/3)))*(h-x)/(d-x) =	3.42E-05
B= (1+2*(acr-c)/(h-x)) =	2.160726195
W=(A-e2)*3*acr/(B) =	-0.1455 OK

LA Z = d - (x/3) =	184.52	
Steel Tensile Stress fs = Ms/(z As) =	6.680	M Pa
Conc Comp Stress = 2Ms/(z b x) =	0.098	M Pa
CHECK STRESS LEVELS		
0.45 fcu =	13.5 M Pa	OK
0.8 fy =	332 M Pa	OK

BASE SLAB 280 THICK M300

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



Loading =

DL = 700
water = 5110
plaster = 42

5852 Kg/m^2

Refer table 22, IS 456/2000, Interior Panel

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

- ve BM = 0.032 X 5852 X 4.328^2 = 3508 Kgm

+ve BM = 0.024 X 5852 X 4.328^2 = 2631 Kgm

Net -ve BM at the face of support=

$$= 3508 - \frac{(3508 + 2631) \text{ X } 4 \text{ X } (4.328 - 0.3) \text{ X } 0.15}{4.328\text{^}2} = 2716 \text{ Kgm}$$

SAY = 3508

$$6bt = \frac{2716 \text{ X } 6}{28\text{^}2} = 20.79 \text{ Kg/cm}^2$$ <20 hence OK with steel contribution

$$\text{-ve As} = \frac{1.5 \text{ x}3508000}{0.8 \text{ x } 0.87 \text{ x } 415 \text{ x}230} = 793 \text{ mm}^2$$

$$\text{+ve As} = \frac{1.5 \text{ x}2631000}{0.8 \text{ x } 0.87 \text{ x } 415 \text{ x}230} = 595\ 3\text{m}^2$$

$$\text{Mini As} = (0.3 - \frac{18}{350}) \text{ X } 28 \text{ X } 0.8 = 5.568 \text{ cm}^2$$

Mini As = 0.313 x 14 = 438.2 mm^2

provide at bottom **10 tor @** **85 c/c** bothways in square mesh at bottom, alternate
bar bent up @ **865** from support 9.24 cm^2

provide at top **10 tor @** **170** bothways in sqmesh 4.62 cm^2

-ve As provided = 924 mm^2 OK

+ve As provided = 924 mm^2 OK

$$\text{Hoop in base slab} = \frac{4673}{1500} = 3.12 \text{ cm}^2$$

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

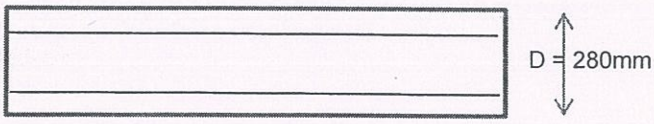
CHECK FOR CRACK WIDTH

Ref Appendix CC3, Wc = 0.1094138 << 0.2 mm OK

CHECK FOR NON CRACKING

Ast=73.92

Asb=57.75



Area of steel provided at top=	(9 - 1) X 924 =	73.92 cm^2
Area of steel provided at bottom=	(1.5 X 9 -1) X 4.62=	57.75 cm^2

As = (28) X 100 + 73.92 + 57.75 = 2931.67 cm^2

$$y = \frac{2800 \times 14 + 73.92 \times 5.5 + 57.75 \times 22.5}{2931.67} = 13.96 \text{ cm}$$

$$I = \frac{100 \times 28^3}{12} + (2800)*(14 - 13.96)^2 + 73.92 \times 8.46^2 + 57.75 \times 8.54^2$$

= 182933.3 + 4.48 + 5290.57 + 4211.8

= 192440 cm^4

Z = 13785.1136 cm^3

$$6bt = \frac{271600}{13785.1136} = 19.71 \text{ Kg/cm}^2 < 20 \text{ hence OK}$$

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

DATA

Permissible crackwidth		0.2 mm	Ms =	35.08 kNm			
Width of the section	b	1000 mm	Mu=	52.62 kNm			
Overall Depth of the section	D	280 mm	Cover	45 mm			
Es		**2.00E+05 Mpa**					
fck		30 Mpa	Ec = 5000*sqrt(30)=	27386.13 Mpa		As req=	660.2
Use 10 dia bars @ 85 mm c/c			*(For calculation of crackwidth use 0.5 Ec)*				
As = 924.00 sqmm per 1000mm	Fy =	415 Mpa				total steel	924 sqmm
a_{cr} = 60.62						Spacing	85 c/c
d=(D-c- 230.00 mm	(CHECK)					Eq Dia	10.00001 cm

r = 0.004017391 , Modulus of elasticity m = 14.60593

(x/d) = -mr+sqrt(mr(2+mr)) = 0.289 ie x= 66.47 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 0.000502918

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 0.000754377

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)) = 1.1463216

W=(A-e2)*3*acr/(B) = 0.1094138 OK

LA,Z = d - 207.84

Steel Tensile Stress fs 182.663 M Pa

Conc Comp Stress = 2 5.078 M Pa

CHECK STRESS LEVELS

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

BASE BEAM 300 X 900 M300 span= 4.328 m

$$\text{Triangular load} = \frac{5852 \times \pi \times 3.06^2}{4} = 43037 \text{ Kg}$$

$$\text{udl} = 0.3 \times 0.62 \times 2500 = 465 \text{ Kg/m}$$

$$\text{SF} = \frac{43037 + 465 \times 4.328}{2} = 22525 \text{ Kg}$$

$$\text{-veBM} = \frac{5}{48} \times 43037 \times 4.328 + \frac{465 \times 4.328^2}{12} - \frac{22525 \times 0.45}{3}$$

$$= 19403 + 726 - 3379$$

$$= 16751 \text{ Kgm}$$

+veBM = 0.6 X 19403 + 0.5 X 726 = 12005 Kgm

$$\text{-veAs} = \frac{1675100}{0.87 \times 1500 \times 84.5} = 15.2 \text{ cm}^2$$

$$\text{+veAs} = \frac{1200500}{0.87 \times 1900 \times 84.5} = 8.595 \text{ cm}^2$$

$$\text{As min} = \frac{0.85}{415} \times 30 \times 84.5 = 5.193 \text{ cm}^2$$

Provide at bottom	2 X 20 tor str				
	2 X 20 tor	extra in centre piece of length	3030		12.57 Sqcm
				OK	
Provide at top	2 X 20 tor str				
	3 X 20 tor	extra near supp upto	1015		15.71 Sqcm
				OK	

$$\text{tau v} = \frac{22525}{30 \times 84.5} = 8.89 \text{ Kg/cm}^2$$

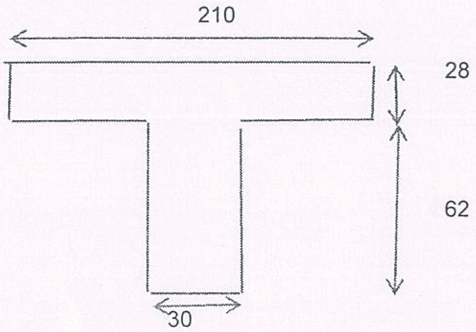
$$\frac{100\text{ As}}{\text{b d}} = \frac{15.708 \times 100}{30 \times 84.5} = 0.620$$ Hence tau c = 3.34 Kg/cm^2

Net SF = 22525 - 3.339 X 30 X 84.5 = 14060 kg

$$\text{8 tor stps @ } \frac{1.005 \times 1750 \times 84.5}{14060} = 10.58 \text{ say } 105$$

supp to	1470	8 tor	stps @	105 c/c
1470	to centre	8 tor	stps @	200 c/c

NON CRACKING CHECK



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

$$= \frac{0.7 \times 4.328}{6} + 0.3 + 6 \times 0.28 = 2.485 \quad m$$

$$bf\ available = \frac{2.485}{2} + 3.06 \times (1 - .707) = 2.13908\ m$$

say 210 cm

$$A = 5880 + 1860 = 7740\ cm^2$$

$$Y = \frac{5880 \times 14 + 1860 \times 59}{7740} = 24.814 \quad cm$$

$$I = \frac{210}{12} \times 28\text{^}3 + 5880 \times 10.8\text{ ^}2 + \frac{30}{12} \times 62\text{^}3 + 1860 \times 34.19\text{^}2$$

$$= 384160 + 687622 + 595820 + 2E{+}06 = 3841352\ cm^4$$

$$Z = \frac{3841352.09}{24.814} = 154805.8\ cm^3$$

$$6bt = \frac{1200500}{154805.83} = 7.76\ \text{OK}$$

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 5540 deep
It is tied at top by roof slab and at bottom by base slab

Roof Slab = 17254 Kg
Self Wt = 49124 Kg
Gallery = 13798 Kg

TOTAL 80176 Kg

$$WR = \frac{80176}{2 \times 3.1415} = 12761\ Kg$$

$$WR2\ (2a) = \frac{12761 \times 3.1416 \times 3.06}{2} \quad 61338$$

-ve BM = 0.137 X 61338 = 8404 Kgm

+ve BM = 0.07 X 61338 = 4294 Kgm

Torsion = 0.021 X 61338 = 1289

$$SF\ at\ support = 12761 \times \frac{3.141593}{4} = 10023\ Kg$$

$$SF\ at\ contra = 12761 \times \left(\frac{3.141593}{4} - 0.336\right) = 5735\ Kg$$

$$6bt = \frac{840400 \times 6}{20 \times 554^2} = 0.8214626\ Kg/cm^2$$

< 18 hence OK

Clause 28, IS 456/1978, for deep beam

$$\frac{L}{D} = \frac{2 \times 3.142 \times 3.06}{4} \times \frac{1}{5.54} = \frac{4.81}{5.54} = 0.868 < 2$$

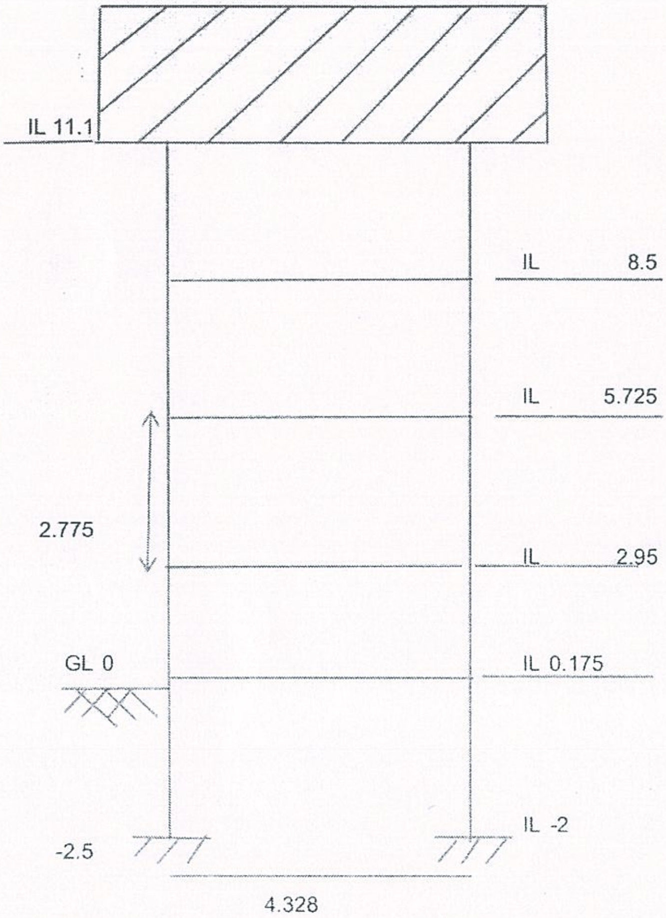
Hence Z = 0.2 X (4.81 + 2 X 5.5) = 3.178 m

$$As = \frac{840400}{1500 \times 317.8} = 1.77\ cm^2$$ very small

$$Net\ SF = 5735 + \frac{1.6 \times 1289}{0.2} = 16047\ Kg$$

$$Tau\ v = \frac{16047}{20 \times 549} = 1.47\ Kg/cm^2$$ very small
Hence O K

Staging - consider 450 dia columns brace 250 350



COLUMN **450** 4 Nos

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

$$I = \frac{3.1416 \times 45^4}{4} = 3220623.3 \text{ cm}^4$$

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

$E = 5000 \times \text{sqrt}(25)$ 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 5.1 X2500 X 0.2 =	49124
Base Slab	=	3.1416 X 3.16^2 X 700 =	21960
Base Beam	=	0.186 X 4.328 X 2500 X 4 =	8050
Gallery	=	3.1416 X (4.16^2 - 3.16^2) X 600 =	13798
water	=	3.14159 X 2.94^2 X 5110 =	138760
plaster	=	(3.1416 X 2.95^2+2 X3.14X3 X 5.11) X 2080X0.02	5078

		TOTAL =	**254024** Kg

WEIGHT OF STAGING

column dia = 400 brace= 250 X 350 span= 4.328

staging ht= 12

Column	=	0.1590 X 4 X 13.6 X 2500 =	21630
Brace	=	4 X 4 X 0.0875 X 3.928 X 2500 =	13748

		TOTAL =	**35378** Kg

ADD FOR STAIR

MB1 = 8.0000 X 2236 = 17888

Total Stg wt = 35378 + 17888 = **53266** 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= 254024 - 9252 = **244772** Kg

Deduct LL on Stair = $\frac{300}{932}$ X 53266 = 17146 Kg

Net Staging Load = 53266 - 17146 = **36120** Kg

SEISMIC ANALYSIS

PARAMETER	STAGING		
	12 M		
DEFLECTION	0.02352		
A) TANK FULL			
1. W=	(244772+(36120/3)) Kg		
=	256812		
2. d AT TOP=	(256812/10) X 0.02352 m		
=	0.6040		
3. PERIOD T=	2 X 3.14 X SQRT(d/9.81) sec		
=	1.5589		
Refer Clause 6.4.5 of IS 1893 for medium strata		For T > 0.55	
4. Sa/g=	**1.67/T =** 1.0713		
5. ah=	(Z/2) (I/R) (Sa/g)	z=	0.16
=	0.042852	I=	1.5
=	0.042852	R=	3
6. SEISMIC FORCE = W X ah Kg			
=	**<u>11005</u>**		
WT OF WATER	138760		
B) TANK EMPTY			
1. W=	(256812-138761) Kg		
=	118052		
2. d AT TOP=	(118052/256812)*0.6041m		
=	0.2777		
3. PERIOD T=	2 X 3.14 X SQRT(d/9.81) sec		
=	1.0571		
REFER FIG 2, IS - 1893/1984, 5 % CRITICAL DAMPING			
4. Sa/g=	**1.67/T =** 1.5798		
5. ah=	(Z/2) (I/R) (Sa/g)	z=	0.16
=	0.063192	I=	1.5
=	0.063192	R=	3
6. SEISMIC FORCE = W X ah Kg			
=	**<u>7460</u>**		

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 = 15.080) = 0.971

K3 = 1.0 (Topography factor)

Hence Vz = 37.87 m/sec

Design wind pressure $P_z = 0.6 \times 37.867^2 = 860\ \text{N/m}^2 = 86.0\ \text{Kg/m}^2$

From appendix D, Reynold's number

$$R = \frac{D\,V_d}{Y} = \frac{6.32 \times 37.87}{1.46 \times 10^{-5}} = 16{,}391{,}933$$

and E/D = 0.001

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

Hence wind on container = Cf Ae pd

$= 0.81162 \times 6.32 \times 6.16 \times 86.0$

$=$ *2719* Kg

Column :

$$R = \frac{D\,V_d}{Y} = \frac{0.45 \times 37.87}{1.46 \times 10^{-5}} = 1{,}167{,}147$$

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

$= 0.9018 \times 11.1 \times 86 \times 4 \times 0.45$

$=$ 1550 Kg

$$\text{Hence equivalent wind at top} = \frac{1550}{2} = 775\ \text{Kg}$$

$$\textbf{Braces :} = \frac{3 \times 1.36 \times 0.35 \times 3.878 \times 2 \times 86.0}{2}$$

= 478 Kg

Total wind force at top = 2719 + 775 + 478

= 3971 Kg

Design of column-

Critical load is	seismic		
	11005		
a) HORZ DESIGN	seismic		
FORCE (Kg)	11005		

ALLOWABLE STRESSES CAN BE INCREASED BY 1.333

b) Mult Factor	11005/1000		
	1.1005		
REFER APPENDIX -I			
Max BM in Col	1.1005 X		
@ Bot Kg-m	5080		
M1=	5590		
@ Top Kg-m	732		
M2=	806		
	1.1005 X		
Additional Axial	5470		
Load in Col Kg	6020		
Vert Seismic	Seismic force/ (2 X 4)		
Force/Col ' Kg' =	1376		
Max BM in Brace	1.1005 X		
Kg - m	-6530		
=	-7186		
Max Story Drift =	0.00664 X	1.100490782 =	0.007307259 m
Panel =	2.775 P=		307290 Kg
		Base Shear=	11005

STABILITY INDEX = $\dfrac{0.00731 \text{ X } 307290}{2.77500 \text{ X } 11005}$ **=0.07352813 <<< 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' (Area of Column 'Ac' cm^2=	**1590**
PARAMETER	STAGING		
	12 m		
a) WT OF CONTAINER	254024		
b) NET WT OF CONT	244772		
c) WT OF STAGING	53266		
d) AXIAL LOAD/COL'Kg'	(Wt Of Cont + Wt Of Staging)/4		
=	76823	Non seismic	
e) Ac Required ' cm^2' =	Axial Load/74.7		
=	1028.41		
f) As Required 'cm^2' =	Ac x 0.008		
=	8.227	0.517 OK	

g)As Provided

Dia of Bar (TOR)	12
No of Bars	10
Area 'cm^2' "Asp"	11.31

Provide 8 TOR links at 150 c/c

h)Area of Compo Sec

Ag=Ac+(1.5x11-1)xAsp 1765.73

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

= 257457.10

Z=I x 2/d = 11442.54

INTERACTION CHECK

Axial Load seismic

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

= 81905

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

46.39

R(Shear in col)=(M1+M2)/(LA) LA "m"= 2.775

R= 2304.88

Depth of Footing = 0.35

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

Net BM = 5321.59

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

46.51

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

(6o,cal/60)+(6b,cal/85)= **1.3202**

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	768 KN		819 KN
Factored load			
Pu= 1.5P	1152 KN	Pu= 1.2 P	983 KN
BM	----		53.22 KNm
Factored BM	----	Bmu=1.2M	63.86 KNm
Pu / fck D^2 =	0.2276		0.1941
Mu / fck D^2	----		0.03
P / fck (Chart 56)			Negligible

Mini concrete area= 11523 / 10.17 1133 cm^2 < 1590 O K

Provide **10 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 = **7186** Kgm M = **7186** Kgm

$$R = \frac{7186 + 7186}{4.328} = 3321 \text{ Kg}$$

Self wt = 218.75 Kg m

$$SF = 3320.8 + \frac{218.75 \times 4.328}{2} = 3794.172 \text{ Kg}$$

$$\text{Net BM} = 7186.2 + \frac{218.8 \times 4.328^2}{12} - \frac{3794 \times 0.45}{3}$$

$$= 7186.2 + 341.461 - 569.1 = 6958.54 \text{ Kg m}$$

$$As = \frac{6{,}958.54 \times 100}{2300 \times 24} = 12.61 \text{ Sq cm}$$

Provide at top and bottom 2 x 20 tor str

Extra at top and bot 3 x 20 tor near support upto 1015 OK As pro 15.707

$$\text{Tau v} = \frac{3794.17}{0.25 \times 31.5} = 4.82 \text{ Kg/cm2}$$

$$\frac{100 \text{ As}}{bd} = 1.995 \qquad \text{Tau c} = 4.698 \text{ Kg/sqcm}$$

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 =	**76823** kg						
SBC =	10 (T/m^2)	Col size	b =	0.318 m	d =	0.318	9.17
Axial Load=	76822.5 Kg	A fact=	1.12				1.5
Area req=	8.60412 m^2	Size req =	2.9333 beq	0.653423295			
Provide sq foot of size =	3 m, A pro=		9 > A req OK Else		a=	3	
Design pressure =	8535.8333 Kg/m^2 < SBC OK				c=	3	
SF=	34337.12 Kg	Offset =	1.340901 m				
BM=	23021.34 Kg-m	Cover	5 cm at ends				
d req =	61.98453 cm	D=	67.48453	**DIA**		**10**	
Provide footing of depth	**70** cm red to		**20** cm at ends				
As =	17.51405 cm^2	As req	22.29953 x		**10** tor		
So =	28.61069 cm	As req	9.107065 x		**10** tor		
Asmin=	18.15005 cm^2	As req	23.10931 x		**10** tor		
Provide bothways at bo	23.109312	**Say 24 x**	**10** tor				
Provide	**3000 x**	**3000 x**		**700 /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 =	**76823**	kg							
SBC =	15	(T/m^2)	Col size	b =	0.318 m		d =	0.318	9.17
Axial Load=	76822.5	Kg	A fact=	1.1					1.5
Area req=	5.63365	m^2	Size req =	2.3735	beq	0.578423295			
Provide sq foot of size =	**2.4**	m, A pro=		5.76	> A req OK Else		a=	2.4	
Design pressure =	13337.24	Kg/m^2 < SBC OK					c=	2.4	
SF=	33318.59	Kg	Offset =	1.040901	m				
BM=	17340.68	Kg-m	Cover	5	cm at ends				
d req =	57.17754	cm	D=	62.67754		**DIA**		**10**	
Provide footing of depth	**70**	cm red to		**20**	cm at ends				
As =	13.19234	cm^2	As req	16.79697	x		**10** tor		
So =	27.76202	cm	As req	8.836925	x		**10** tor		
Asmin=	14.73005	cm^2	As req	18.75484	x		**10** tor		
Provide bothways at bo	18.754843		**Say 20 x**	**10**	tor				
Provide	**2400**	**x**	**2400 x**			**700 /200 thk footing**			

Footing for column as per Pg 410 of Jain & Jaikrishna

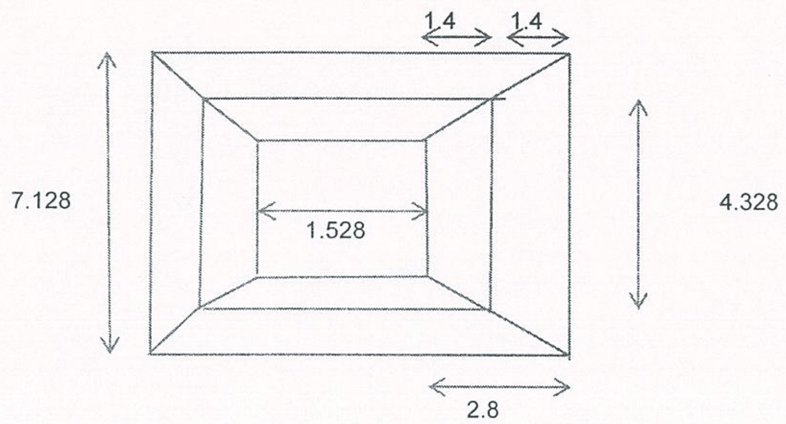
FOOTING FOR COLUMNS 4 Nos			***FOR SBC***	***20***	***(T/m^2)***				
GRADE OF CON		CAST IN M-250, DESIGN IN M-250					**450 dia**		
Max axial load =	**76823**	kg							
SBC =	20	(T/m^2)	Col size	b =	0.318 m		d =	0.318	9.17
Axial Load=	76822.5	Kg	A fact=	1.1					1.5
Area req=	4.225238	m^2	Size req =	2.0555	beq	0.540923295			
Provide sq foot of size =	**2.1**	m, A pro=		4.41	> A req OK Else		a=	2.1	
Design pressure =	17420.068	Kg/m^2 < SBC OK					c=	2.1	
SF=	32591.07	Kg	Offset =	0.890901	m				
BM=	14517.71	Kg-m	Cover	5	cm at ends				
d req =	54.0999	cm	D=	59.5999		**DIA**		**10**	
Provide footing of depth	**65**	cm red to		**20**	cm at ends				
As =	11.98869	cm^2	As req	15.26444	x		**10** tor		
So =	29.47684	cm	As req	9.382768	x		**10** tor		
Asmin=	12.29459	cm^2	As req	15.65393	x		**10** tor		
Provide bothways at bo	15.653927		**Say 16 x**	**10**	tor				
Provide	**2100**	**x**	**2100 x**			**650 /200 thk footing**			

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

Wt of ESR = 254024 + 35378 = 289402 Kg

span = 4.328 m

$$\text{Area reqd} = \frac{289402 \times 1.1}{7500} = 42.45 \text{ m}^2$$



Provide strip raft as shown in fig

Width of strip = 2.8 m

$A_{provided}$ = 7.128^2 - 1.528^2 = 48.4736 > A reqd OK

Design of Raft Slab Balance Cantilever = 1.4 - 0.15 = 1.25 m

$$p = \frac{289402}{48.4736} = 5971 \text{ Kg/m}^2$$

SF = 5971 X 1.25 = 7463.75 Kg

BM = 7463.75 X 0.625 = 4665 Kgm

Dreqd = sqrt(4665/11.14) = 22.56 cm

Provide 350 depth reducing to 200 at ends

$$As = \frac{466500}{0.87 \times 2300 \times 30.5} = 7.644 \text{ cm}^2$$

As min = 0.0012 X 30.5 X 100 = 3.66 cm^2

Provide at botttom 10 tor @ 100 c/c str bars OK As pro 7.853

Provide distribution - 8 X 10 tor @ on each side

Provide 4 X 12 tor diagonal bars at bottom corners

Raft Beam 300 X 900 span 4.328 m

Load UDL = 5971 X 1.4 X 2 = 16719 Kg/m

$$SF = \frac{16719 \times 4.328}{2} = 36180 \text{ Kg}$$

$$\text{-ve BM} = \frac{16719 \times 4.328^2}{12} - \frac{36180 \times 0.45}{3}$$

= 26098 - 5427 = 20671 Kgm

$$\text{-ve AS} = \frac{2067100}{0.87 \times 2300 \times 84.5} = 12.23 \text{ cm}^2$$

+ve BM = 0.5X 26098 = 13049 Kgm

$$\text{+ve AS} = \frac{1304900}{0.87 \times 2300 \times 84.5} = 7.72 \text{ cm}^2$$

$$\text{As min} = \frac{0.85}{415} \times 30 \times 84.5 = 5.2 \text{ cm}^2$$

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	1015

$$\frac{100\,As}{b\,d} = \frac{12.32 \times 100}{30 \times 84.5} = 0.486$$ Hence tau c = 3.06 Kg/cm^2

Net SF = 36180 - 3.0552 X 30 X 84.5 = 28435 kg

$$\text{10 tor stps @} \quad \frac{1.5708 \times 2300 \times 84.5}{28435} = 10.74 \text{ say } 100$$

supp to	1100	10 tor	stps @	100 c/c
1100	to centre	10 tor	stps @	200 c/c

STAIRCASE : M250

Total Rise =	2.775	From GL to LDL total F	12 m	
		Rise per flight =	2.775 m	
span =	4328			
		As per NIT		
		1) Width of Flight =	1000	mm
		2) Flight of Rise =	2.775 m	
		3) no of risers =	18	nos
		4) Rise =	154.167	mm
		5) No of treads =	17	nos
		6) Tread =	254.589	

span = 4328 m

$$\text{Tread} = \frac{4328}{17} = 254.589 \text{ m}$$

WAIST SLAB : 150 THK 1000 wide span = 4328

$$\text{Load self} = \frac{375}{0.8554} = 439 \text{ Kg/m}^2$$

$$\text{Steps} = \frac{0.154167}{2} \times 2500 = 193 \text{ Kg/m}^2$$

LL = 300 Kg/m^2

Finish = 0 Kg/m^2

----

932 Kg/m^2

$$SF = \frac{932 \times 4.328}{2} = 2017 \text{ Kg}$$

$$BM = \frac{932 \times 4.328^2}{12} = 1455 \text{ Kgm}$$

d = 150 - 25 - 5= 120 mm

Mu = 1455 X 1.5 = 2182.5 Kgm

As = 6.3 cm2/m

So As total= 6.3 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 =	2017	2017	Kg/m^2

		2236	Kg/m^2

SF = 2236 Kg

$$BM = \frac{2236 \times 1^2}{2} = 1118 \text{ Kgm}$$

Mu = 1677 Kgm

d = 35 - 2.5 + 1.0 = 31.5

As = 1.85 cm2

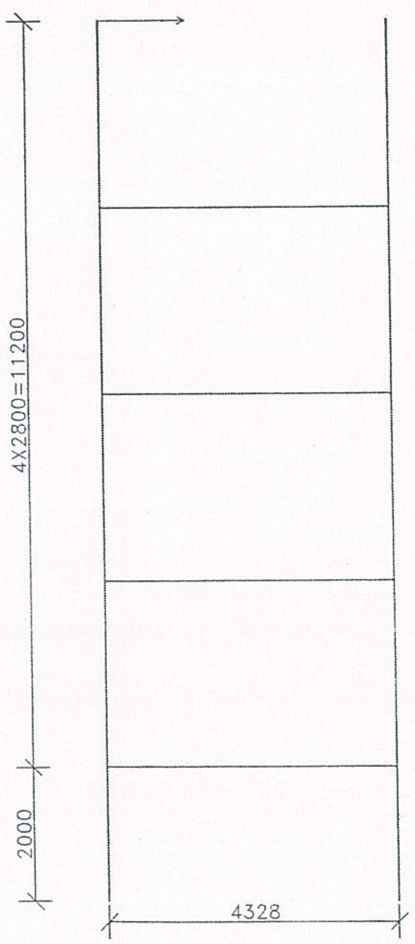
Provide at top	2x	20 tor str	15.708 sqcm
	3x	20 tor str	
At bottom	2x	20 tor str	15.708 sqcm
	3x	20 tor str	

stps - 8 tor @ 100 c/c

125 kl – 12 m staging MP

Load 1: seis, Nodal Connectivity

SCALE = 1:81	UNITS: kg m	DATE:31-05-18



125 kl - 12 m staging MP
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	0.00168	0.00005	-0.0013118
4	0.00168	-0.00005	-0.0013118
5	0.00726	0.00010	-0.0019924
6	0.00726	-0.00010	-0.0019924
7	0.01390	0.00014	-0.0020700
8	0.01390	-0.00014	-0.0020700
9	0.02006	0.00015	-0.0016480
10	0.02006	-0.00015	-0.0016480
11	0.02332	0.00015	0.0000000
12	0.02332	-0.00015	0.0000000
MAX.	0.02332	0.00015	-0.0020700
NODE	11	9	7

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

Bm.	*Node*	*Axial*	*V2*	*M3*
1	3	0.000	-1812.673	-3922.625
	4	0.000	1812.673	-3922.625
2	5	0.000	-2733.891	-5916.139
	6	0.000	2733.891	-5916.139
3	7	0.000	-2819.873	-6102.204
	8	0.000	2819.873	-6102.204
4	9	0.000	-2216.543	-4796.599
	10	0.000	2216.543	-4796.599
5	1	-9582.979 T	2500.000	5800.622
	3	9582.979 T	-2500.000	-800.622
6	3	-7770.306 T	2500.000	4723.247
	5	7770.306 T	-2500.000	2276.753
7	5	-5036.416 T	2500.000	3639.386
	7	5036.416 T	-2500.000	3360.614
8	7	-2216.543 T	2500.000	2741.590
	9	2216.543 T	-2500.000	4258.411
9	9	0.000	2500.000	538.188
	11	0.000	-2500.000	6461.810
10	2	9582.979 C	2500.000	5800.622
	4	-9582.979 C	-2500.000	-800.622
11	4	7770.306 C	2500.000	4723.247
	6	-7770.306 C	-2500.000	2276.753
12	6	5036.416 C	2500.000	3639.385
	8	-5036.416 C	-2500.000	3360.614
13	8	2216.543 C	2500.000	2741.590
	10	-2216.543 C	-2500.000	4258.411

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

Bm.	*Node*	*Axial*	*V2*	*M3*
14	10	0.000	2500.000	538.188
	12	0.000	-2500.000	6461.810
MAXIMUM		9582.979 C	-2819.873	-6461.810
Beam no.		10	3	14

Verified
3/8/18

प्राध्यापक
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