OFFICE OF THE EXECUTIVE ENGINEER CITY DIVISION -I ,Ajmer

Name of work: **Providing, laying, jointing of Rising main for New proposed OHSR at Trambay Station with distribution pipelines in Tarkash ki Bagichi , New Chandmari and other areas under City Sub Division- I, Ajmer**

PROJECT DETAILS

1. Name of Scheme	: Providing, laying, jointing of Rising main for New proposed OHSR at Trambay Station with distribution pipelines in Tarkash ki Bagichi , New Chandmari and other areas
2. Name of Covered District	: Ajmer
3.Population 2016	: 1440
4. Design Year	: 2046
5.Population 2046 Forecast	: 2413
6. Rate of Supply	: 135 LPCD +15% losses in lines=155 LPCD.(as per guidelines)
7. Design Demand	: 374 KLD
8. Proposed OHSR Capacity required 50% of Design Demand	: 190 KL
9. Proposed Rising Main	: 150 mm DI K-7
10. Proposed Distribution	: 150 mm DI K-7
11. Total Estimated Cost	: 176.83 Lacs

Assistant Engineer
PHED City sub Dn I Ajmer

Executive Engineer
PHED City Dn I Ajmer

Design of Rising Main

From: **Tramway** To: **Harijan Basti SR**

1) Water requirement :	Year	Peak Discharge	
A. Initial	2016	0.22	mld
B. Intermediate	2031	0.29	mld
C. Ultimate	2046	0.37	mld
2) Pumping main	LENGTH	300	M
3) Static head for pump	ST.HEAD	51.83	M
4) Design period	YEAR	30	yr.
5) Combined eff. of pump set	EFF. %	65	%
6) Cost of pumping unit	Rs./KW	12000	Rs
7) Interest rate	INTEREST	10.00	%
8) Life of electric motor & pump set	P.Yrs	15	yr.
9) Energy charges per kWh	P/KWH	550	paise
10) Pumping hours for discharge at the end of 15 years	hours	4	hrs

Pipe Dia in mm	Material	Class	HWC	Rate Rs/m
150	DI	K7	140	1382
200	DI	K7	140	1729
250	DI	K7	140	2255
300	DI	K7	140	2867
350	DI	K7	140	3412
400	DI	K7	140	4077
450	DI	K7	140	4793
500	DI	K7	140	5688
600	DI	K7	140	7529
700	DI	K7	140	9934
800	DI	K7	140	13007
900	DI	K7	140	15948

CALCULATIONS:	1st 15 years		2nd 15 years	
1) Discharge at Start OF PERIOD	0.22	mld	0.29	mld
2) Discharge at the end of 15 yrs	0.29	mld	0.37	mld
3) Flow End of 15 year	20	lps	26	lps
4) Average Discharge	0.26	mld	0.33	mld
5) Avg.pumping hours during the period	3.54	hrs	3.54	hrs
6) KW required at combined efficiency of pumping set	0.30	* H1	0.39	* H2
7) Annual charges for energy Rs.	7121	* KW1	7121	* KW2

Static level difference	
RL Tramway	509.60
RL Harijan Basti SR	535.43
DIFFERENCE	25.83
STAGING +WAT. COL.(7)	25.00
Terminal head	1.00
TOTAL	51.83

Modified Hazen William's Formula

V= 143.534CR r0.6575 S0.5525

h= [L(Q/CR)1.81]/[994.62D4.81]

Friction Head Loss (First 15 years)

Dia. in mm	L	Q	CR	Q/CR	(Q/CR)1.81	994.62	D	D4.81	h(First 15 yrs)
150	1000	0.020	1.000	0.020	0.0008	994.620	0.150	0.0001089	7.812
200	1000	0.020	1.000	0.020	0.0008	994.620	0.200	0.0004345	1.958
250	1000	0.020	1.000	0.020	0.0008	994.620	0.250	0.0012708	0.669
300	1000	0.020	1.000	0.020	0.0008	994.620	0.300	0.0030546	0.278
350	1000	0.020	1.000	0.020	0.0008	994.620	0.350	0.0064116	0.133
400	1000	0.020	1.000	0.020	0.0008	994.620	0.400	0.0121873	0.070
450	1000	0.020	1.000	0.020	0.0008	994.620	0.450	0.0214759	0.040
500	1000	0.020	1.000	0.020	0.0008	994.620	0.500	0.0356489	0.024
600	1000	0.020	1.000	0.020	0.0008	994.620	0.600	0.0856855	0.010
700	1000	0.020	1.000	0.020	0.0008	994.620	0.700	0.1798546	0.005
800	1000	0.020	1.000	0.020	0.0008	994.620	0.800	0.3418715	0.002
900	1000	0.020	1.000	0.020	0.0008	994.620	0.900	0.6024298	0.001

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Velocity

Dia. in mm		CR	r=A/P=D/4	r0.6575	S	S0.5525	V
150	143.534	1.000	0.038	0.115	0.008	0.069	1.135
200	143.534	1.000	0.050	0.139	0.002	0.032	0.639
250	143.534	1.000	0.063	0.162	0.001	0.018	0.409
300	143.534	1.000	0.075	0.182	0.000	0.011	0.284
350	143.534	1.000	0.088	0.202	0.000	0.007	0.209
400	143.534	1.000	0.100	0.220	0.000	0.005	0.160
450	143.534	1.000	0.113	0.238	0.000	0.004	0.126
500	143.534	1.000	0.125	0.255	0.000	0.003	0.102
600	143.534	1.000	0.150	0.287	0.000	0.002	0.071
700	143.534	1.000	0.175	0.318	0.000	0.001	0.052
800	143.534	1.000	0.200	0.347	0.000	0.001	0.040
900	143.534	1.000	0.225	0.375	0.000	0.001	0.032

Friction Head Loss (Second 15 years)

Dia. in mm	L	Q	CR	Q/CR	(Q/CR)1.81	994.62 D		D4.81	h(Second 15 yrs)
150	1,000.000	0.026	1.000	0.026	0.001	994.620	0.150	0.000109	12.465
200	1,000.000	0.026	1.000	0.026	0.001	994.620	0.200	0.000434	3.124
250	1,000.000	0.026	1.000	0.026	0.001	994.620	0.250	0.001271	1.068
300	1,000.000	0.026	1.000	0.026	0.001	994.620	0.300	0.003055	0.444
350	1,000.000	0.026	1.000	0.026	0.001	994.620	0.350	0.006412	0.212
400	1,000.000	0.026	1.000	0.026	0.001	994.620	0.400	0.012187	0.111
450	1,000.000	0.026	1.000	0.026	0.001	994.620	0.450	0.021476	0.063
500	1,000.000	0.026	1.000	0.026	0.001	994.620	0.500	0.035649	0.038
600	1,000.000	0.026	1.000	0.026	0.001	994.620	0.600	0.085686	0.016
700	1,000.000	0.026	1.000	0.026	0.001	994.620	0.700	0.179855	0.008
800	1,000.000	0.026	1.000	0.026	0.001	994.620	0.800	0.341871	0.004
900	1,000.000	0.026	1.000	0.026	0.001	994.620	0.900	0.602430	0.002

Velocity

Dia. in mm	143.534	CR	r=A/P=D/4	r0.6575	S	S0.5525	V
150	143.534	1.000	0.038	0.115	0.012	0.089	1.470
200	143.534	1.000	0.050	0.139	0.003	0.041	0.827
250	143.534	1.000	0.063	0.162	0.001	0.023	0.529
300	143.534	1.000	0.075	0.182	0.000	0.014	0.367
350	143.534	1.000	0.088	0.202	0.000	0.009	0.270
400	143.534	1.000	0.100	0.220	0.000	0.007	0.207
450	143.534	1.000	0.113	0.238	0.000	0.005	0.163
500	143.534	1.000	0.125	0.255	0.000	0.004	0.132
600	143.534	1.000	0.150	0.287	0.000	0.002	0.092
700	143.534	1.000	0.175	0.318	0.000	0.001	0.067
800	143.534	1.000	0.200	0.347	0.000	0.001	0.052
900	143.534	1.000	0.225	0.375	0.000	0.001	0.041

TABLE 1 - VELOCITY AND HEADLOSSES FOR DIFFERENT PIPE SIZES

S No	Pipe Size in mm	friction head loss per 1000 m		Velocity in m/sec	Velocity in m/sec	other losses at 10%	total losses (H1) including static head	Friction head loss in total pipe length	other losses at 10%	total losses (H2) including static head
		1st stage flow	2nd stage flow	1st stage flow	2nd stage flow	1st stage flow	1st stage flow	300.00		
							52		2nd stage flow	
1	150	7.81	12.47	1.14	1.47	0.78	54.41	3.74	0.37	55.94
2	200	1.96	3.12	0.64	0.83	0.20	52.48	0.94	0.09	52.86
3	250	0.67	1.07	0.41	0.53	0.07	52.05	0.32	0.03	52.18
4	300	0.28	0.44	0.28	0.37	0.03	51.92	0.13	0.01	51.98
5	350	0.13	0.21	0.21	0.27	0.01	51.87	0.06	0.01	51.90
6	400	0.07	0.11	0.16	0.21	0.01	51.85	0.03	0.00	51.87
7	450	0.04	0.06	0.13	0.16	0.00	51.84	0.02	0.00	51.85
8	500	0.02	0.04	0.10	0.13	0.00	51.84	0.01	0.00	51.84
9	600	0.01	0.02	0.07	0.09	0.00	51.83	0.00	0.00	51.84
10	700	0.00	0.01	0.05	0.07	0.00	51.83	0.00	0.00	51.83
11	800	0.00	0.00	0.04	0.05	0.00	51.83	0.00	0.00	51.83
12	900	0.00	0.00	0.03	0.04	0.00	51.83	0.00	0.00	51.83

TABLE 2 - KILOWATTS & COST OF PUMP SETS REQUIRED FOR DIFFERENT PIPE SIZES AND PIPE COST

S No	Pipe Dia in mm	class of Pipe	H1 total head in meters	KW reqd plus 100% stand by	pump cost at Rs per Kw	H2 total head in meters	Kw required plus 100% stand by	Pump cost at Rs per KW	cost of pipe per meter	total cost of pipe in thousand Rs
			1st stage flow in MLD		0.29	2nd stage flow in MLD			0.37	
					12000			12000		300
1	150	K7	54.41	33	395	55.94	44	526	1382	415
2	200	K7	52.48	32	381	52.86	41	497	1729	519
3	250	K7	52.05	32	378	52.18	41	491	2255	677
4	300	K7	51.92	31	377	51.98	41	489	2867	860
5	350	K7	51.87	31	377	51.90	41	488	3412	1024
6	400	K7	51.85	31	377	51.87	41	488	4077	1223
7	450	K7	51.84	31	377	51.85	41	488	4793	1438
8	500	K7	51.84	31	377	51.84	41	487	5688	1706
9	600	K7	51.83	31	376	51.84	41	487	7529	2259
10	700	K7	51.83	31	376	51.83	41	487	9934	2980
11	800	K7	51.83	31	376	51.83	41	487	13007	3902
12	900	K7	51.83	31	376	51.83	41	487	15948	4784

TABLE 3 - COMPARATIVE STATEMENT OF OVERALL COST OF PUMPING MAIN FOR DIFFERENT PIPE SIZES

	1st stage flow		0.29	mld	2nd stage flow		0.37	mld		
S No	Cost of pump set	Annual Energy Charges	capitalized energy cost	Total capitalised cost	Cost of pump set	Annual Energy Charges	capitalized energy cost	Initial capital investment for pumps+Annual energy	Pipe Dia	Grand total cost first and second stage
	Thousand Rs	Thousand Rs	Thousand Rs	Thousand Rs	Thousand Rs	Thousand Rs	Thousand Rs	Thousand Rs	mm	Thousand Rs
1	395	117	892	1702	526	156	1187	410	150	2,112
2	381	113	860	1,760	497	147	1,122	388	200	2,148
3	378	112	853	1,908	491	146	1,107	383	250	2,290
4	377	112	851	2,088	489	145	1,103	381	300	2,469
5	377	112	850	2,251	488	145	1,101	381	350	2,631
6	377	112	850	2,450	488	145	1,101	380	400	2,830
7	377	112	850	2,664	488	145	1,100	380	450	3,044
8	377	112	850	2,933	487	145	1,100	380	500	3,313
9	376	112	850	3,485	487	145	1,100	380	600	3,865
10	376	112	850	4,206	487	145	1,100	380	700	4,586
11	376	112	850	5,128	487	145	1,100	380	800	5,508
12	376	112	850	6,011	487	145	1,100	380	900	6,391

Minimum Capitalized cost ! 2,112 -

Optimum pipe size corresponds to minimum capitalized cost

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M/s RIDHI SIDHI CONSTRUCTION

TITTLE	Design of Over Head Service Reservoir		Date	REV NO.
			29-May-17	0

PROVIDING, LAYING JOINTING TESTING AND COMMISSIONING OF RISING MAIN AND DISTRIBUTION PIPELINE AND CONSTRUCTION OF RCC OHSR OF 190 KL AND 18 M STAGING AT TRAMBAY, CHANDMARI, TARKASH KI BAGICHI AND VARIOUS AREAS UNDER CITY DIVISION- I, AJMER INCLUDING DEFECT LIABILITY PERIOD OF ONE YEAR

PLACE : **TRAMBAY**

DOCUMENT NO.

CAPACITY	=	190.000 m^3
STAGING HEIGHT	=	18.000 m
SOIL BEARING CAPACITY	=	9.500 t/m^2
DEPTH OF FOUNDATION	=	2.000 m
SEISMIC ZONE	=	II
BASIC WIND SPEED	=	47.000 m/sec
Type of soil	=	Soft Soil
Damping	=	5%

DESIGN OF ELEVATED SERVICE RESERVOIR

Basic Data :

Capacity of Reservoir	=	190.000 m^3	
Staging Height above GL	STA=	18.000 m	(As per the tender conditions)
Soil Bearing Capacity	SBC=	9.500 t/m^2	(As per the tender conditions)
Depth of foundation	FDN=	2.000 m	
Seismic Zone	=	II	(As per the tender conditions)
Wind Speed in the zone	=	47.000 m/s	(As per the tender conditions)

Assumed size of various componets of SR

The diemnsions of the tank are assuemed as unedr :-

Internal Diameter of water tank	IDT =	8.30 m	
Height tank above conical wall	HTW =	2.80 m	
Free Board (including beam depth)	FB =	0.30 m	(As per tender condition)
Horizontal span of cone wall	HSCW =	1.10 m	
Vertical span of cone wall	VSCW =	1.25 m	
Base dia. of bottom dome	DBD =	5.70 m	
Rise of top dome	RTD =	1.45 m	
Rise of bottom dome	RBD =	0.85 m	
Lower edge dia. of con. wall	DCWb	6.10 m	
Thickness of top dome	TTD =	0.15 m	
Top ring beam width	TRBW =	0.25 m	
Top ring beam depth	TRBD =	0.20 m	
Thickness of vertical wall at Top	TWT =	0.15 m	
Thickness of vertical wall at bottom	TWB =	0.15 m	
Thickness of vertical wall (Average)	TWA =	0.15 m	
Balcony beam total width	BBW =	1.05 m	(including wall width)
Balcony depth at edge	BBWE =	0.15 m	
Balcony depth at junction	BBWJ =	0.15 m	
Balcony beam depth (Average)	BBD =	0.15 m	
Thickness of Conical Dome	TCD =	0.20 m	
Thickness of Bottom dome	TBD =	0.20 m	
Number of columns	NC =	6 No.	
Dia. of Columns	DC =	0.45 m	
Width of ring beam over columns	WRBC =	0.45 m	
Depth of ring beam over columns	DRBC =	1.00 m	
DIA OF COLUMN CENTRES	DCC=	5.65	
No. of Braces	NB =	4 No.	

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Width of braces	WB =	0.25 m	
Depth of braces	DB =	0.50 m	
Depth of Foundation Beam	DFB =	0.90 m	
Width of Foundation ring beam	WFB =	0.45 m	
Height of Dead Storage	HDS =	0.15 m	
Depth of Foundation Slab at face of FB	Ds=	0.40 m	
Depth of Foundation Slab at outer edge	Dse=	0.30 m	

LOADS :

Live Loads : 0.132

Live load on top dome	LLTD =	2.000 KN/m^2	
Live load on balcony beam	LLBB =	2.000 KN/m^2	
Live load on stairs	LLST=	2.000 KN/m^2	
R1 OF TOP DOME =ROUND(((IDT/2)^2+RTD^2)/2/RTD,2)	RaTD	6.660 m	RaTDd=
R2 OF BOT DOME =ROUND(((DBD/2)^2+RBD^2)/2/RBD,2)	RaBD	4.400 m	
THETA OF TOP DOME =ASIN(IDT/2/RaTD)	Xt	0.673 RAD	38.545 deg
THETA OF BOT DOME =ASIN(DBD/2/RaBD)	Xb	0.632 RAD	36.222 deg

Dead Loads :

Considerd only the self weight of the structure

Wind Loads :

Calculated considering the basic wind speed as 47.00 m/s

Seismic Loads :

The zone has been considerd in the II zone and accordingly the loads have been calculated.

Minimum % of Reinforcement as per clause 8.1 of IS:3370 Part 2: ptm= 0.24%

Grade of concrete used :

					Modular Ratio m
For top dome & ring beam	fck=	30.000 N/mm^2	np=	0.418	9.33
For vertical wall		30.000 N/mm^2	np=	0.418	9.33
For balcony cum ring beam		30.000 N/mm^2	np=	0.418	9.33
For conical dome and bottom dome		30.000 N/mm^2	np=	0.418	9.33
For ring beam over column		30.000 N/mm^2	np=	0.418	9.33
For all type columns		30.000 N/mm^2	n=	0.289	9.33
For bracings		30.000 N/mm^2	n=	0.289	9.33
For foundations		30.000 N/mm^2	n=	0.289	9.33
	fst=	415.000 N/mm^2			

TABLE SHOWING COVERS AND LAP LENGTHS

ITEM	TYPE OF BAR	LAP LENGTH	CLEAR COVER in mm	
			Water face	Other face
Top & bot. dome	Sq. mesh	30*D	45.000	30.000
	Hoops	50*D	45.000	30.000
Edge beam	Hoops	50*D	35.000	30.000
Vertical wall	Verticals	30*D	45.000	30.000
	Hoops	50*D	45.000	30.000
Cone Top Platform	Hoops	50*D	45.000	30.000
	mesh	50*D	35.000	30.000
Cone wall	Vertical	30*D	45.000	30.000
	Hoops	50*D	45.000	30.000
Main Beam	Top face	38*D	45.000	-
	Bottom face	47*D	-	30.000
	Side Face	47*D	-	30.000
Columns	Main Beam	33*D	-	40.000
Brace	Top & bottom	47*D	-	30.000
Foundation	All Type	50*D	-	50.000
Steel for all RCC work = Fe 415 grade				

CALCULATION FOR CAPACITY OF TANK

Internal dia of tank wall		= 8.300	m	
Height of the tank wall above cone wall excluding free board		= 2.800	m	
Volume of water in Cylindrical portion		= 151.497	m^3	
PI()*IDT^2*HTW/4				
Horizontal span pf cone wall		= 1.100	m	
Vertical span of cone wall		= 1.250	m	
Volume of water in conocal portion				
Volume upto top of con. wall		= 51.290	m^3	
PI()/12*VSCW*(IDT^2+DCWb^2+IDT*DCWb)				
Volume of water in bottom dome portion				
Dia of bottom dome		= 5.200	m	
DCWb - 2 * WRBC				
Rise of bottom dome		= 0.850	m	
Radius of bottom dome (Top face)		RaBD= 4.400	m	
0.5*(BDBD^2*0.25/RBD+RBD)				
Radius of bottom dome (Center line)		RaBD= 4.750	m	
Cos θ		= 0.810		
(4.4-0.85)/4.4				
		Xb 36.222	degree	
		< 51.800	degree	
The dome is in compression throught				
Volume below bottom dome		= -9.347	m^3	
=-PI()*(3*BDBD^2/4+RBD^2)*RBD/6				
Volume of dead storage		1.647	m^3	
=(PI()*(HDS^2*((TAN(Xc))+(1/(TAN(Xb))))/2*DCC))+(PI()*DCC*WRBC*HDS)				
Any other deduction in capacity (Internal column & beam)		= 0.573	m^3	
Actual volume provided =	Wwc=	191.221	m^3	
	>	190.000	m^3	O.K.

DESIGN OF TOP DOME

Thickness of to dome adopted		= 0.150	m	
Diameter at base of dome		= 8.300	m	
Rise of lower face of top dome		= 1.450	m	
Radius of top Dome		RaTD = 6.660	m	
Rise of center line of top dome		Rtd= 1.525	m	
=RTD+0.5*TTD				
Radius of center line of top dome		Ratd= 6.735	m	
Span of the wall at center line		IDTc1= 8.450	m	
Span of the dome at center line		IDTc2= 8.375	m	
COS(Xt)		= 0.782		
(RaTD-RTD)/RaTD				
θt	Xt=	38.530	degree	
		< 51.800	degree	
Therefore, the dome is in compresion throught				
Self weight of the dome		= 3.750	KN/m^2	
TTD* 25.0				
Live Load on the top dome		= 2.000	KN/m^2	
Total Load on the top dome		Wt= 5.750	KN/m^2	
max NcT		NcT= 0.1433	KN/m^2	
=Wt*RaTD/TTD/(1+COS(Xt))/1000				
max NhT		NhT= 0.128	N/mm^2	
=Wt*RaTD/TTD/2/1000	<	6.000	N/mm^2	O.K.

Both the max. stresses are compressive and within limits hence min. temp. reinf. @ 0.24 % of gross sectional area shall be provided as per (IS3370,PartII).

% of reinforcement. required	= 0.240	%
Area of reinforcement required 0.24*0.15*1000*1000/100	= 360.000	mm^2
Diameter of bars to be provided	= 8.000	mm
Spacing of bars required both ways 1000*(PI()/4)*8^2/360	= 140	mm c/c
Spacing to be provided	= 140.000	mm c/c

The reinforcement shall be provided in the form of sq. mesh maintaining the required clear cover

From continuity analysis

Moment at edges due to continuity	= -1.670	KN-m
Thickness of dome reqd. at edges SQRT(ABS(-1.66954748900006)*60000/18)	= 70.772 Adopted thickness is correct	mm
Area of steel reqd. for moment 1.67*1000000/(0.861*(0.15*1000-50-(8/2))*130)	= 155.375	mm^2/m
Diameter of bars to be provided	= 8.000	mm
Spacing of bars required 1000*(PI()*8^2/4)/155.374963611835 Provide	= 330.000 = 150.000	mm c/c mm c/c
Hoop tension at edges (from continuity analysis)	= -39.359	KN
Steel required for hoop force ABS(-39.359)*1000/130	= 302.759	mm^2
Dia. of circumfrential bars provided at top	= 8.000	mm
Number of bars required 302.759*4/(PI()*12^2)+1	= 6.000 Say 8.000	Nos. Nos.

Vertical bars of outer face of wall shall be extended in dome for a length of900 mm as radial bars

Check for shear -

Total weight of top dome =ROUND(2*PI()*Ratd*Rtd*TTD*25,2)	= 227.540	KN
Total live laod :=ROUND(2*PI()*Ratd*Rtd*2,2)	= 121.350	KN
Total load	= 348.890	KN
Load per metr of periphery	= 13.143	KN
Shear stress develpoed in dome	= 0.131	N/mm^2
	< 0.800	N/mm^2

Provided thickness of dome is safe in shear

SUMMARY		
a) Grade of concrete	= M	30.000
b) Grade of Steel	= Fe	415.000
c) Thickness of dome at crown	= 150.000	mm
d) Thickness of dome at edges	= 150.000	mm
e) Rise of top dome(lower face)	= 1.450	m
f) Radius of top dome (lower face)	= 6.660	m
g) Reinforcement		
Sq mesh	8 mm @ 140.000	mm c/c In both direction
Radial reinforcement at top edge	8 mm @ 150.000	mm c/c measured at spring
Circumfrential reinf. At top	8 mm @ 8.000	Nos in a length of 845 mm
h) Quantity of concrete	= 9.102	m^3
i) Total weight of top dome	= 227.540	KN
j) Total live load on top dome	= 121.350	KN

DESIGN OF TOP RING BEAM

Internal diametr of the beam	= 8.300	m	
Mer. stress causing hoop tension	= 0.143	N/mm^2	
Thickness of dome adopted	= 0.150	m	
Cos θt	COS(Xt)= 0.782	deg.	
Hoop Tension at the edges 0.1433*1000*0.15*0.782*4.225	= 69.767	KN	
Adopted size of beam			
Width of beam	= 0.250	m	
Depth of beam	= 0.200	m	
Hoop force at the junction	= 12.966	KN	(From continuity analysis)
Hoop force for design purpose 69.766643925+	= 69.770	KN	
Area of steel reqd. for the hoop 69.77*1000/130	= 536.690	mm^2	
Diameter of bars to be provided	= 12.000	mm	
Number of bars required 536.69*4/(PI()*12^2)	= 4.745 Say 5.000	Nos. Nos.	
Stress induced in concrete 69.77*1000/((0.25*0.2)*1000000+(9.33-1)*5*PI()*12^2/4)	= 1.275 < 1.500	N/mm^2 N/mm^2	O.K.
Provide 5 numbers 12 mm dia. Bars			
Provide 2 legged stirrups of	8.000	mm dia. bars	
Spacing of the stirrups to be provided	= 110.000	mm c/c	

As the beam is continously supported

SUMMARY		
a) Grade of concrete	= M	30.000
b) Grade of Steel	= Fe	415.000
c) Width of beam	= 250.000	mm
d) Depth of beam	= 200.000	mm
e) Reinforcement		
Circumfrential reinforcement	12 mm @ 5.000	Nos
Links	8 mm @ 110.000	mm c/c all along beam
f) Quantity of concrete	= 1.343	m^3
g) Total weight of ring beam	= 33.576	KN

DESIGN OF VERTICAL WALL

Internal diameter of tank	= 8.300	m
Water depth for design purpose	= 3.100	m
HTW+FB	(Including Free Board)	
Net Height of wall	= 3.100	m

Dist(h-y) from top (M)	y/h	water pressure(cal) (Kg/m2)	Provide Thickness (mm)	H^2/Dt			
0.000	1.000	0.000	150.000	7.719			
0.310	0.900	310.000	150.000	7.719			
0.620	0.800	620.000	150.000	7.719			
0.930	0.700	930.000	150.000	7.719			
1.240	0.600	1240.000	150.000	7.719			
1.550	0.500	1550.000	150.000	7.719			
1.860	0.400	1860.000	150.000	7.719			
2.170	0.300	2170.000	150.000	7.719			
2.480	0.200	2480.000	150.000	7.719			
2.790	0.100	2790.000	150.000	7.719			
3.100	0.000	3100.000	150.000	7.719			

Dist from top (M)	Distance from bottom (M)	Hoop static force (IS TAB) (KN)	Hoop static force(cal) (KN)	Hoop static force(max) (KN)	Steel Required (mm^2)	Min. Thickness required (mm)	Provide Thickness (mm)	Remarks
0.000	3.100	-1.204	0.000	39.359	302.759	23.414	150.000	O.K.
0.310	2.790	11.531	13.100	13.100	100.769	7.793	150.000	O.K.
0.620	2.480	24.594	26.200	26.200	201.538	15.586	150.000	O.K.
0.930	2.170	37.975	39.290	39.290	302.231	23.374	150.000	O.K.
1.240	1.860	51.444	52.390	52.390	403.000	31.167	150.000	O.K.
1.550	1.550	64.557	65.490	65.490	503.769	38.960	150.000	O.K.
1.860	1.240	74.407	78.590	78.590	604.538	46.753	150.000	O.K.
2.170	0.930	76.908	91.680	91.680	705.231	54.540	150.000	O.K.
2.480	0.620	66.934	104.780	104.780	806.000	62.333	150.000	O.K.
2.790	0.310	40.378	117.880	117.880	906.769	70.127	150.000	O.K.
3.100	0.000	0.000	130.980	130.980	1007.538	77.920	150.000	O.K.
From Continuity Analysis								
	0.000	39.359	0.000	39.359	302.759	23.414	150.000	O.K.
	3.100	148.053	0.000	148.053	1138.873	88.077	150.000	O.K.

Steel Required

Dist. from top (m)	Distance from bottom (m)	Provided thickness (mm)	Steel required mm^2	Min. steel required mm^2	Hoop steel to be provided mm^2	Dia. of bars (mm)	Spacing reqd. for both faces (mm c/c)	Spacing recomended in mm
0.000	3.100	150.000	302.759	360.000	360.000	8.000	279.253	150.000
0.310	2.790	150.000	100.769	360.000	360.000	8.000	279.253	150.000
0.620	2.480	150.000	201.538	360.000	360.000	8.000	279.253	150.000
0.930	2.170	150.000	302.231	360.000	360.000	8.000	279.253	150.000
1.240	1.860	150.000	403.000	360.000	403.000	8.000	249.457	150.000
1.550	1.550	150.000	503.769	360.000	503.769	8.000	199.558	150.000
1.860	1.240	150.000	604.538	360.000	604.538	8.000	166.294	150.000
2.170	0.930	150.000	705.231	360.000	705.231	8.000	142.551	140.000
2.480	0.620	150.000	806.000	360.000	806.000	10.000	194.888	150.000
2.790	0.310	150.000	906.769	360.000	906.769	10.000	173.230	150.000
3.100	0.000	150.000	1138.873	360.000	1138.873	10.000	137.926	130.000

Average thickness adopted	= 150.000	mm
H^2/Dt	= 7.719	
(HTW+FB)^2/(IDT*TWA)		

AS PER TABLE 10 OF IS:3370 (PART IV)

Dist. from top, H (M)	Coeff. C	Moment CwH^3 (KN-M)	Min. Thickness Required (mm)	Thickness Provided (mm)	Steel Required for B.M. (mm^2)	Minimum steel reqd. at each face (mm^2)	Steel required at
0.000	0.000	0.000	0.000	150.000	0.000	180.000	outer face
0.310	0.000	0.000	0.112	150.000	0.000	180.000	outer face
0.620	0.000	0.000	0.957	150.000	0.027	180.000	outer face
0.930	0.000	0.002	2.619	150.000	0.202	180.000	outer face
1.240	0.001	0.018	7.389	150.000	1.610	180.000	outer face
1.550	0.002	0.068	14.278	150.000	6.011	180.000	outer face
1.860	0.003	0.196	24.277	150.000	17.378	180.000	outer face
2.170	0.004	0.407	34.941	150.000	35.999	180.000	outer face
2.480	0.003	0.442	36.428	150.000	39.128	180.000	outer face
2.790	-0.002	-0.536	40.092	150.000	-47.394	180.000	inner face
3.100	-0.015	-4.521	116.463	150.000	-420.759	180.000	inner face

From Continuitiy Analysis

Top	1.659	70.555	150.000	134.772	180.000	outer face
Bottom	-3.431	101.449	150.000	-303.464	180.000	inner face

Provided Vertical steel as under

Maximum reinforcement reqd inner side = 420.759

Inner Face

Dia = 8.000 mm

Spacing = 110.000 mm c/c Provide 110.000 mm c/c

Curtail bars after 1.0 m from base = Not required Vertical bars Through mm

Spacing = Not required Vertical bars Through mm c/c

Maximum reinforcement reqd outer side = 180.000

Outer Face

Dia = 8.000 mm

Spacing = 279.253 mm c/c Provide 150.000 mm c/c

SUMMARY

a) Grade of concrete	= M	30.000
b) Grade of Steel	= Fe	415.000
c) Thickness of wall at top	= 150.000	mm
d) Thickness of wall at bottom	= 150.000	mm
e) Quantity of concrete	= 12.344	m^3
f) Total weight of vertical wall	= 308.603	KN

DESIGN OF RING BEAM CUM BALCONY BETWEEN VERTICAL & CONICAL WALL

Internal diameter	= 8.300	m	
Water Depth for Design purpose	= 3.100	m	
Width	= 1050.000	mm	
Average depth	= 150.000	mm	
Load coming on beam			
Top Dome			
Dead load =2*PI()*RaTD*RTD*TTD*25	= 227.538	KN	D.L.
Live load on top dome =2*PI()*RaTD*RTD*LLTD	= 121.353	KN	L.L.
Top Ring Beam =PI()*(IDT+TRBW)*TRBD*(TRBW)*25	= 33.576	KN	D.L.
Vertical wall =PI()*TWA*(IDTc)*(HTW+FB)*25	= 308.603	KN	D.L.
Load of ring beam cum balcony			
Dead load =PI()*(IDT+BBW)*(BBW-twb)*BBD*25+0.15*0.15*PI()*(IDT+(2*BBW+TWB))*25	= 100.727	KN	D.L.
Live load	= 148.990	KN	L.L.
	7.510		
Total Load	= 948.297	KN	
Load per Running Length 948.297/(PI()*(IDT+TWb))	= 35.722	KN/m	
Meridional Thrust in Conical Dome 35.722*COS(xc)	= 47.599	KN/m	
Horz. Component causing Hoop Tension	= 31.457	KN/m	

Hoop Tension due to this Force HT1 31.457*IDT/2		= 132.906	KN/m	
Hoop Tension due to water pressure HT2 =9.8*(HTW+FB)*(BBWJ)*IDTc/2		= 19.253 =	KN/m	
Total Hoop Tension = HT1 + HT2 132.905825+19.253325		152.159	KN/m	
Hoop Tension from cont. analysis		= 137.998	KN/m	
Hoop Tension for Design		= 152.159	KN/m	
Steel required 152.15915/130*1000		= 1170.455	mm^2	
Provide Hoop bars	Dia	= 12.000	mm	
	Nos.	= 11.000		
		say 11.000	Nos.	
Stress in Concrete 152.15915*1000/(0.9*0.15*1000000+10.0*11*PI()*12^2/4)		= 1.032	N/mm^2	
		< 1.500	N/mm^2	O.K.
Design as Balcony				
Dead Load =BBWJ*25		= 3.750	KN/m^2	
Live Load		= 2.000	KN/m^2	
Total		= 5.750	KN/m^2	
Overhang =BBW-TWB+TCD/2		= 1.000	m	
Bending Moment 5.75*1^2/2		= 2.875	KN-m	
Minimum depth required		= 97.895	mm	
SQRT(3.75*1000*6/(18/10))		< 150.000	mm	O.K.
Steel area required 2.875*1000000/(jp*(BBWJ*1000-30-12/2)*130)		= 225.313	mm^2/m	
Minimum steel required =(pim*100*10*BBWJ*1000		= 360.000	mm^2/m	
Steel required on one face		= 225.313	mm^2/m	
Provide Stirrups	**Dia**	= 8.000	mm	
	Legged	= 2.000		
	Required spacing	= 223.092	mm c/c	
	Provided spacing	= 150.000	mm c/c	

Since the Balcony is acting as a beam also, it's section should not be cut at any place.
A projection should be provided to accommodate staircase / ladder

SUMMARY

a)	Grade of concrete	= M	30.000
b)	Grade of Steel	= Fe	415.000
c)	Thickness of beam at edge	= 0.150	mm
d)	Thickness of beam at support	= 0.150	mm
e)	Quantity of concrete	= 4.626	m^3
f)	Total weight of balcony beam	= 108.238	KN
g)	Live load on balcony beam	= 148.990	KN

DESIGN OF CONICAL DOME

Thickness	= 200.000	mm		
Internal Dia	= 8.300	m		
Cone Wall				
Horizontal span	= 1.100	m		
Vertical span	= 1.250	m		
Dia at Lower edge of Cone wall	= 6.100	m		
Water depth for Design purpose	= 3.100	m		
Slant Height	SLc= 1.665			
Weight of water above conical wall =PI()*(IDT-HSCW)*(HTW+FB+VSCW/2)*HSCW*9.8	= 908.296	KN	34.834	
Selfweight based on average dia =PI()*TCD*(IDT+DCWb+2*TCD)*(HSCW^2+VSCW^2)^0.5/2*25	= 193.536	KN		
Total	= **1101.834**	KN		
Superimposed Load	= 948.297	KN		
Total Vertical Load 948.297+1101.834	= 2050.131	KN		
Load per Running length 2050.131/(PI()*6.1	= 106.980	KN/m	ws*tan(Xc)	32.026
Angle of cone wall from vertical ATAN(VSCW/HSCW)	Xc= 0.722	radians		
Meridional Thrust in Con. Dome 106.98/COS(0.722)	= 142.548	KN/m		
Meridional stress 142.548/(1000*0.2)	= 0.713	N/mm^2	O.K.	
	< 6.000		O.K.	

Hoop Tension at different depth from top of conical dome

Depth from top of Conical dome (M)	Hoop Tension (KN)	Hoop Tension from Cont. analysis (KN)
0.000	186.267	197.405
0.250	189.215	-
0.375	190.150	-
0.500	190.725	-
0.625	190.941	-
0.750	190.798	-
0.875	190.296	-
1.000	189.434	-
1.125	188.213	-
1.250	186.632	5.656

Hoop Tension for design purpose (maximum from the table)	= 197.405	KN		
Steel Required per m 197.404596728179*1000/130	= 1518.497	mm^2		
Provide Hoop bars Dia	= 12.000	mm		
Required spacing	= 148.960	mm c/c		
Provided spacing	= 148.000	mm c/c	each face	
No. of bars provided on each face	= 11.250		say 12.000	Bars
Hoop steel provided per m	= 1630.232	mm^2		
Stress in Concrete 197.404596728179*1000/(1000*0.2*1000+(i95-1)*1630.23186348443)	= 0.920	N/mm^2		
	< 1.500	N/mm^2	O.K.	
Load per unit length (2050.131/(PI()*(8.3-1.1)))/1.1	= 82.396	KN/m		
Span	= 1.100	m		
Bending moment 82.396*1.1^2/12	= 8.308	KN-m		
Area of steel required 8.308*1000000/(0.861*(0.2*1000-35-8/2)*130)	= 508.390	mm^2	Provided=	946.834
Bending Moment from cont. anal.	= 13.218	KN-m		
Tensile stress due to bending	= 1.877	N/mm^2		
Compressive stress due to thrust	= 0.713	N/mm^2		
Net stress	= 1.164	N/mm^2		
	< 2.000	N/mm^2	O.K.	
SIGMA-CC/SIGMA-CC+SIGMA-CBC/SIGMA-CBC	0.353			
	< 1.000		O.K.	
SIGMA-CC/SIGMA-CC+SIGMA-CBT/SIGMA-CBT	0.800			
	< 1.000		O.K.	

Thickness Provided	= 200.000	mm
Area of steel required at bottom end 13.218*1000000/(0.861*(0.2*1000-50-8/2)*130)	= 808.847	mm^2
Minimum steel required :=ptm*100*10*TCD*1000	= 480.000	mm^2

Provide Longitudinal steel as under

On upper side(at water face) Dia	= 8.000	mm		
Require Spacing at bottom end	= 62.145	mm c/c		
Provided Spacing	= 150.000	mm c/c		
	(Measured at top end of conical dome)			
Spacing of bars available at lower end	= 110.241	mm		
Provided area of reinforcement	= 455.960	mm^2	O.K.	
Provided additional bars extending in bottom dome	10.000	mm dia @	160.000	mm c/c
Total area of steel provided at lower end	= 946.834	mm^2		
	Provided reinforcement is safe.			

On lower side

Adopted dia of bars	= 8.000	mm
Required spacing	= 209.440	mm c/c
Provided spacing of bars	**= 150.000**	**mm c/c**
	(Measured at top end of conical dome)	

SUMMARY

a) Grade of concrete	= M	30.000
b) Grade of Steel	= Fe	415.000
c) Thickness of conical wall	= 200.000	mm
d) Reinforcement		
-Circumfrential reinforcement	12 mm @ 148.000	mm c/c all along
-Vertical reinforcement		
On water face	8 mm @ 150.000	mm c/c measured at top
Additional bars	10 mm @ 160.000	mm c/c all extended in bottom dome
On outer face	8 mm @ 150.000	mm c/c measured at top
e) Quantity of concrete	= 7.532	m^3
f) Total weight of conical wall	= 188.307	KN

DESIGN OF BOTTOM DOME

Rise		=	0.850 m
Dia at base		=	5.200 m
Radius of Bottom dome		=	4.400 m
Cosθ	Cos(Xb)=		0.807
θ		=	36.222 degrees
Thickness of bottom dome		=	200.000 mm
Dead Load of dome 0.2*1000*25.0/1000		=	5.000 KN/m^2
Total dead load		=	117.495 KN
Water Load on dome (PI()*bdbd^2*(HTW+FB+VSCW)/4+-9.347)*9.8		=	813.740 KN
Surface Area of dome =2*PI()*RaBD*(RBD)		=	23.499 m^2
Projected area		=	35.500
Water Load per sqm 813.74/23.499		=	34.629 KN/m^2
Total Distributed Load 5+34.629		=	39.629 KN/m^2
Max. maridional stress 39.629*RaBD/TBDe/(1+COS(Xb))/1000	Ncb=		0.483 N/mm^2
Max. Hoop stress 39.629*RaBD/TBD/2/1000	Nhb=		0.436 N/mm^2
Maridional Force 0.483*200		=	96.600 KN/m

Stresses are within permissible limits and only min. steel is required

Minimum Reinforcement required =ptm*100*TBD*10000	= 480.000	mm^2

Provide as mesh reinforcement dia	=	10.000 mm	
required spacing	=	163.625 mm c/c	
provide spacing	=	160.000 mm c/c	
Moment at edges	=	-7.500 KN-m	From Continuity Analysis

Thickness needed SQRT(ABS(-7.5)*60000/18)		= 150.000	mm		
At edge provide thickness	TBDe=	200.000	mm	Up to 540.000	mm
Actual	TBDe=	200.000			
Steel needed 7.5*1000000/(jp*(200-35-(10/2))*130)		= 462.111	mm^2		
Min. steel required 0.0024*TBDe*1000000		= 480.000	mm^2		

- Provide additional radial bars at edges in a length of 780 mm

Adopted dia of bars	=	10.000	mm		
Required spacing	=	160.000			
Provided spacing of bars	=	160.000	mm c/c	Provided spacing is correct	
Hoop Tension	=	6.307	KN	From Cont. Analysis	
Steel needed ABS(6.30656616634692*1000/130)	=	48.512	mm^2		
Provided Circumferential bars dia .	=	8.000	mm		
Number of bars required 48.512/2/(PI()*8^2/4)+1	=	1.483	Say	4.000	Nos.

SUMMARY

a) Grade of concrete		= M	30.000
b) Grade of Steel		= Fe	415.000
c) Thickness of dome at crown		= 200.000	mm
d) Thickness of dome at edges		= 200.000	mm
e) Rise of bottom dome upper face)		= 0.850	m
f) Radius of bottom dome (top face)		= 4.400	m
g) Reinforcement			
Sq. mesh in both direction	10 mm @	160.000	mm c/c
Radial reinf. at top, edge	10 mm @	160.000	mm c/c measured at spring of dome
Circumferential reinf at top	8 mm	4.000	Nos in 870 mm length
h) Quantity of concrete		= 4.700	m^3
i) Total weight of top dome		= 117.496	KN
j) Total weight of water on **dome**		= 813.740	KN

DESIGN OF RING BEAM OVER COLUMNS

Adopted size of the ring beam

Depth	=	1.000 m		
Width	=	0.450 m		

Superimposed Load on Ring Beam :

Top Dome	=	227.540 KN	D.L.	
	=	121.350 KN	L.L.	
Top Ring Beam	=	33.576 KN	D.L.	
Vertical Wall	=	308.603 KN	D.L.	
Balcony Ring beam	=	108.238 KN	D.L.	
	=	148.990 KN	L.L.	
Conical wall	=	188.307 KN	D.L.	2050.131
Bottom Dome	=	117.496 KN	D.L.	931.235
				340.507
Water Load on the dome 191.221*10	=	2074.525 KN		
Total	=	3328.623 KN		3321.873
Number of Columns	=	6.000		
Column circle diameter C/C	=	5.650 m		
Width of Ring beam	=	450.000 mm		
Depth of Ring Beam	=	1000.000 mm		
C/C Span of beam =PI()*DCC/NC	=	2.958 m		
Clear span of the beam	=	2.508 m		
Effective span of the beam	=	2.884 m	FRAME C/C	
Span / depth ratio 2.8842*1000/1000	=	2.900		
	>	2.500	Hence design as Ordinary Beam	
Self weight =WRBC*PI()*DCC*drbc*25	=	199.687 KN		
Additional weight due to shape etc.	=	5.000 KN		
Grand Total of weights	=	3533.311 KN		

Load per Running Length w 3533.311/(PI()*DCC)	=	199.060 KN/m	For staad Exl LL LL	189.218 KN/m 173.988 15.230	
Angle subtended by one part of beam on the centre of columns	Xr=	30.000 degrees			
Moment Coefficients C1	=	-0.089			
C2	=	0.045			
C3	=	0.009			
Point of Contraflexure is at z	=	12.750 degrees			
Radius R =DCC/2	=	2.825 m			
Meridional thrust in conical dome	=	142.548 KN/m	HORIZ COMP	94.208	
Hoop Force at edges 142.548*SIN(Xc)*DCWb/2	=	287.335 KN		Compressive	
Meridional thrust in bottom dome	=	96.600 KN/m	HORIZ COMP	77.931	
Hoop Force 96.6*Xc*BDBD/2	=	181.338 KN		Tensile	
Net Hoop force	=	105.997 KN		Compressive	
Net Thrust =F733*2/DCC	=	-16.277 KN/m		Inward	-16.277
Hoop stress =F799*1000/(WRBC*DRBC)/1000000	=	0.236 N/mm2			
Torsion due to this thrust -16.277209389117*DRBC*1000/2000	=	8.139 KN-m / m			
Support Moment C1*w*R^2*2θ -0.089*3533.311/(PI()*DCC)*(DCC/2)^2*360/NC*PI()/180	=	148.061 KN-m	As per staad		KN-m
Mid span Moment C2*w*R^2*2θ 0.045*199.06*(DCC/2)^2*360/NC*PI()/180	=	74.862 KN-m	As per staad		KN-m
Twisting Moment C3*w*R^2*2θ	=	14.972 KN-m	As per staad		KN-m
Shear Force at Supprot w*R*θ 199.06*DCC/2*PI()/NC	=	294.440 KN	As per staad		KN
Shear force at Pt. of contraflexure w*R* (θ -) ζ	=	220.832 KN	As per staad		KN
Additional Moment due to torsion MAX(14.972,)*(1+DRBC/WRBC)/1.7	=	28.378 KN-m		Supp	
Moment from Continuity Analysis	=	5.695 KN-m / m			
TotaEffective torsional moments	=	8.139			
This will act as torsion for the beam					
Thrust (Cont. Analysis)	=	7.287 KN	Inward		
Moment in beam due to torsion 8.139*(1+DRBC/WRBC)/1.7	=	15.427 KN-m	from conti. Analysis		
Effective support moment ROUND(148.060608073462+15.426862745098,3)	=	163.487 KN-m			
Effective midspan moment 74.862+15.426862745098	=	90.289 KN-m			
Net shear at support 294.44+28.378*2.958*1.6/(0.45*2/1)	=	375.565 KN			
Net shear at pt. of contraflexure 163.487+(360/2/NC-7.5)*(PI()/180)*DCC*8.139*1.6/(WRBC/DRBC)	=	314.136 KN			

Analysis of beam

Required effective depth of beam SQRT(163.487*1000000/WRBC/1000/1.79949)	=	449.325 mm	
Alternatively	=	990.000 mm	
Provided over all depth of beam as	=	1000.000 mm is	
Effective depth of beam =	=	940.000 mm is	
Lever arm z for the beam (deep)	=	876.840 m	

Lever arm z for the beam (ordinary)	=	809.340				
Adopetd lever arm	=	809.340				
Area of steel required at support	=	1553.849 mm^2				
163.487*1000000/130/940jp						
Mini. Tension steel required	=	666.386 mm^2				
Total steel 0.24%		1080.000 mm^2				
Dia. of bars provided	=	20.000 mm				
Number of bars required at top of beam	=	5.000	Say	**5.000**	Nos.	
Steel reqd. at mid span	=	792.083 mm^2				
Dia. of bars provided	=	16.000 mm				
Number of bars required	=	4.309	Say	5.000	Nos.	
at bottom of beam throughout the length						

As the depth of beam is more than 450 mm, the side face reinforcement shall be provided as per clause no. 26.5.1.7(b) of IS 456

Side face reinforcement	=	**450.000 mm^2**
0.10*DRBC*WRBC*10000		
Provided dia. of bars	=	**12.000 mm**
Number of bars required	=	**3.979 no.**

Provide 12 mm dia. bars 2 nos. on each face of beam

Transverse reinforcement

Shear Reinforcement					
Effective depth	=	940.000 mm			
Shear stress at support	=	0.888 N/mm^2			
375.565*1000/(940*0.45*1000)					
% of steel provided	=	0.349 %	0.250	0.500	0.262
Permissible shear stress	=	0.262 N/mm^2	(As per table 23 of IS 456)		

As permissible shear stress is less than the shear stress at support, shear reinforcement shall be reqd.

Vc	=	110699.554 N	
Vs	=	264865.446 N	
375.565*1000-110699.553509991			
Asv/Sv	=	1.610	
:=F830/F784/175			
Dia. of bars adopted	=	10.000 mm	
Spacing of 2 legged stirrups	=	90.000 mm	
Provided spacing	=	**90.000 mm c/c**	0.610 m FROM BOTH EDGE
Vs at point of contraflexure	=	203436.390 N	9.000 NOS
=ROUND(F778*1000-F828*F825*WRBC*1000,3)			
Asv/Sv	=	1.237	
203436.39/940/175			
Providing dia of bar	=	10.000 mm	0.888 m CENTRAL PORTION
For4 legged stirrups Sv	=	120.000 mm	8.000 NOS 111.000
Provided spacing	=	**120.000 mm c/c**	

Check for combined direct stress and bending stress

Area of the beam	=	0.450 m^2	
Area of steel provided in compression zon	=	1005.310 mm^2	
Area of steel provided in tension zone	=	1570.796 mm^2	
Equivelant area of the beam	=	0.476 m^2	
		0.043	
Sectiom modulus of beam	=	0.085 m^3	
Bending moment	=	163.487 KN m	
Bending Stress	=	1923.376 KN/m^2	
163.487/0.085	=	1.923 N/mm^2	
Direct Stress	=	0.236 N/mm^2	Compressive
Check for combined stresses	=	1.687 N/mm^2	
1.923-0.236	<	2.000 N/mm^2	
$\sigma - CC' \ \sigma - CC \ \sigma - CBc' \ \sigma - CBc=$		0.222	
$(-)\sigma - Ct' \ \sigma - Ct \ \sigma - CBt' \ \sigma - CBt=$		0.93	

SUMMARY

a) Grade of concrete	= M	30.000	
b) Grade of Steel	= Fe	415.000	
c) Size of beam			
- Width of beam	= 500.000	mm	
- Depth of beam	= 1000.000	mm	

d) Reinforcement			
Bars at top		20 mm 5.000	Nos
Bars at Bottom		16 mm 5.000	Nos
Stirrups near support, 2 legged		10 mm @ 90.000	mm c/c
Stirrups at mid span, 2 legged		10 mm @ 120.000	mm c/c
h) Quantity of concrete		= 7.987	m^3
i) Total weight of ring beam		= 199.687	KN

DESIGN OF STAGING

Grade of Concrete used	=	M 30.000	
Adopted dia. Column	=	0.450 m	
Height of Staging	=	18.000 m	
Depth of foundation below GL	FDN=	2.000 m	
Depth of foundation ring beam	DFB=	0.900 m	
Net Column height =STA+FDN-DFB-DRBC-HDS	=	17.950 m	
Number of brace	=	4.000 No	
Straight C/C span of bracing	spBR=	2.825 m	
Clear length of each brace	spBRu=	2.375 m	
Size of braces			
Width	=	0.250 m	
Depth	=	0.500 m	
C/C vertical distance between braces	Lc=	3.590 m	
Clear vertical distance between braces	Lcu=	3.090 m	
C/C vertical distance between lower brace	=	3.590 m	
No. of columns	=	6.000	
Dia of columns	=	450.000 mm	
Dia of columns circle	=	5.650 m	
Load acting on columns at base of ring beam over column 3533.311/6	=	588.885 KN	
Superimposed load per column			
Column load 17.95*PI()*(0.45)^2*25/4	=	71.371 KN	
Load of braces 2.375*WB*DB*25*4	=	29.688 KN	
Load from Staircase	=	48.916 KN	
Load of Platform and piping and int. col.		10.000 KN	
Total	=	748.860 KN	
Compressive stress of conc	=	6.000 N/mm^2	
Compressive stress of steel	=	140.000 N/mm^2	
Unsupported length of the column	=	3.090 m	
Mini. Lateral dimension	=	0.450 m	
Effective length of column	=	3.540	
Ratio of L/D	=	7.867	
	<	12.000	
Hence design as short axially loaded column			
Mini. Eccentricity for design of col.	=	21.180 mm	
=Lcu/500*1000+DC*1000/30	<	22.500 mm	=0.05*DC*1000
Required area of concrete	=	101857.936 mm^2	
Required dia of column	=	360.124 mm	
Provided dia. of column	=	450.000 mm	
Provided area of concrete	=	157836.757 mm^2	
Capacity of concrete alone to take load	=	947020.539 N	
		947.021 KN	
Min. steel required @ 0.8%	=	1272.345 mm^2	
dia	dbc=	16.000 mm	
nos.	=	5.328	Say 6.000 Nos
As	Astc=	1206.372 mm^2	
Spacing of the 8 mm ties	=	190.000 mm	
Provide ties of 8 mm dia. tor steel @		190.000 mm c/c	
Factored load 1.5*748.86*1000	=	1123289.318 N	
Ultimate Load carriaying capacity= 0.4*fck*157836.76+0.67*fst*Astc		2229472.696	
	>	1123289.318 safe	

CALCULATION FOR WIND LOAD

Height of staging	=	18.000 m	
Height of tank	=	23.950 m	
Basic wind speed	WS=	47.000 m/s	
Probability factor K1	Ka=	1.070	

Local topography factor K3		Kb=	1.000
Terrain, Height & size factor for (K2)			
	For tank	K2t=	1.100
	For staging	K2s=	1.100
Force Coeff			
	For tank	Fct=	0.600
	For staging	Fcs=	0.600
Design wind speed			
	For tank =Ka*Kb*K2t*WS	DWSt=	55.319 m/s
	For staging =Ka*Kb*K2s*WS	DWSs=	55.319 m/s
Wind pressure			
	For tank =+Fct*DWSt^2	=	1836.115 N/m²
	For staging =Fcs*DWSs^2	=	1836.115 N/m²
Design wind pressure			
	For tank 0.7*1836.115	=	1285.281 N/m²
	For staging 0.7*1836.115	=	1285.281 N/m²
	for braces		1836.115 N/m²

Member	Projected area	Design wind pressure	Wind . Force	Lever arm over mid of last column	Moment	Lever arm above col. Top	Moment above col. Top
	m^2	N/m^2	KN	(M)	(KN-M)	(M)	(KN-M)
Top dome	8.218	1285.281	10.562	25.567	270.044	6.517	68.834
Ring Beam	1.760	1285.281	2.262	24.500	55.421	5.450	12.328
Wall	26.660	1285.281	34.266	22.850	782.968	3.800	130.209
Beam	1.560	1285.281	2.005	21.225	42.557	2.175	4.361
Con. wall	9.500	1285.281	12.210	20.675	252.445	1.625	19.842
RB over Col	6.100	1285.281	7.840	19.350	151.708	0.500	3.920
Columns	40.095	1285.281	51.533	9.525	490.855		
Brace	12.200	1836.115	22.401	9.525	213.366		
Total			143.079	15.791	2259.364	3.464	239.494

				ht of RBOC
Wind forces on the container portion	= 69.145	KN	81.459	39.916
Lever arm of above forces above top of columns	= 3.464	m	13.576	
The wind force on stagging	= 73.934	KN		
			2.464 KN	

The wind force on stagging shall be considerd to be distributed on panel points. The design load coming on columns in different panels due to vertical loads and due to wind forces are calculated.

Total over turning moment on the tank column base	= 2259.364	KNm	
Total over turning moment on the tank on foundation	= 2388.136		2313.644
	=		
FOR STADD FORCE ON RING BEAM	= 13.493	KN/m	
Additional thrust in lee ward column (Considering farthest column)	= 230.058	KN	
For brace level Ist	= 53.469	KN	

CALCULATION FOR SEISMIC FORCES

Total weight of tank	=	3262.971 KN (Excluding DL,LL and incl water load)
Dead weight of tank	=	1188.447 KN
227.54+33.576+308.603+108.238+188.307+117.496+199.687+5		
Live load on tank	=	270.340 KN
121.35+148.99		
Weight of water	=	1912.206 KN
Weight of staging	=	959.846 KN (Including weight of stairs and landing)
71.371*NC+29.688*NC+48.916*NC+10*6		
MI OF COL		2305796307.647 mm⁴
Ec=		27386127875.258
Stiffness of columns	=	16377570.266
Equivalent spring of the system	=	19653.084 KN/m
16377570.265*NC/(NB+1)/1000		
Lumped weight when tank empty	=	1508.396 KN

(including 1/3rd of stagging weight)

Lumped weight when tank full	=	3582.920 KN
Delta when tank empty	=	0.077 m
Delta when tank empty	=	0.182 m
Fundamental period when tank empty = 2*PI()*SQRT(0.077/9.81)	=	0.557 sec
Fundamental period when tank full 2*PI()*SQRT(0.182/9.81)	=	0.856 sec
Refer fig. 2 of IS 1893 with 5% damping		
Avg. acc. coeff when tank empty	=	2.500
Avg. acc. coeff when tank full	=	1.965
Horz. seismic coeff when tank is		
Empty 0.1*1.5*1*2.5/2/2.5	=	0.075
Full 0.1*1.5*1*1.965/2/2.5	=	0.059

Particulers	Weight KN	Lumped weight KN	Shear coeff	Shear force KN For tank (f) fr seismic water	Lever arm m	Moment KN-m
tank (e)	1458.787	1458.787	0.075	109.409	22.850	2499.996
tank (f)	3262.971	3262.971	0.059	173.517	22.850	3964.865
staging (e)	959.846	319.949	0.075	23.996	9.525	228.563
staging (f)	959.846	319.949	0.059	18.861	9.525	179.651

Shear at base of column (empty)	=	133.405 KN
Shear at base of column (full)	=	192.378 KN
Moment at base of column (e)	=	2728.560 KN-m
Moment at base of column (f)	=	4144.516 KN-m
Forces Considered for Design		
Shear	=	192.378 KN
Moment	=	4144.516 KN-m
Additional thrust in farthest column 4144.516*4/NC/DCC	=	469.028 KN

Design Forces and Permissibel Stresses

Stifness of column	=	16377.570 KN/m	
MI brace	=	2604166666.667	
Ec	=	27386127875.258	
Stifness of brace	=	12653.307 KN/m	
Ratio of stiffness of braces to column	=	0.773 >	0.100
Shear design factor	=	1.400	
Shear per column at base 1.4*192.378/NC	=	44.888 KN	
Bending Moment at base 44.888*Lcu	=	138.704 KN-m	
Equivalent area of section	=	170699.693 mm^2	
Equivalent dia	=	466.199 mm	
Equivalent M.I. of section	=	2305796307.647 mm^4	
Section modulus Z	=	10247983589.541	
Direct stress	=	4.387 N/mm^2	
Bending Stress	=	0.014 N/mm^2	
Age factor	=	1.000	
Concrete Grade used	=	M 30.000	
Permissible stress in comp	=	8.000 N/mm^2	
Permissible stress in bending	=	10.000 N/mm^2	

Forces in Column at Various Brace Levels

Particular	Top of brace (counted from top) Base	4	3	2	1	0
Direct Load						
Tank	3262.971	3262.971	3262.971	3262.971	3262.971	
Columns	385.401	299.757	214.112	128.467	42.822	
Braces	178.125	142.500	142.500	142.500	142.500	
Staircase	318.148	247.448	176.749	106.049	35.350	0.000
Total	4144.645	3952.676	3796.332	3639.987	3483.643	0.000
LOAD PER COL	690.774	658.779	632.722	606.665	580.607	0.000
Axial Thrust	230.058	184.046	138.035	92.023	46.012	0.000
DIR AXIAL THRUST	920.832	842.826	770.757	698.688	626.619	0.000
Wind Load						
Tank	69.145	69.145	69.145	69.145	69.145	0.000
Column	46.380	36.073	25.767	15.460	5.153	-5.153
Braces	22.401	16.800	11.200	5.600	0.000	-5.600
Total	137.926	122.019	106.112	90.205	74.299	-10.753
Seismic Load						
Tank	173.517	173.517	173.517	173.517	173.517	0.000
Columns	13.825	11.060	8.295	5.530	2.765	0.000
Braces	3.500	2.625	1.750	0.875	0.000	-0.875
Total	190.842	187.202	183.562	179.922	176.282	-0.875
Effective Shear for design SHd=	190.842	187.202	183.562	179.922	176.282	-0.875
Shear per column	44.530	43.680	42.831	41.982	41.132	-0.204
Bending Moment	68.799	67.486	66.174	64.862	63.550	-0.315
A equi	170699.693	170700	170700	170700	170700	0
Moment of Inertia	2305796308	2305796308	2305796308	2305796308	#########	0
Direct stress	4.047	3.859	3.707	3.644	3.446	0.000
Bending stress	6.713	6.585	6.457	6.329	6.201	0.000
Per. comp stress inc wind F	10.660	10.660	10.660	10.660	10.660	10.660
Per. bend stress inc wind F	13.330	13.330	13.330	13.330	13.330	13.330
$\sigma_{cc}'/\sigma_{cc} + \sigma_{cbc}'/\sigma_{cbc}$	0.88	0.856	0.832	0.817	0.788	0.000

DESIGN OF BRACES

Concrete Grade used	=	M 30.000
Width of brace	=	0.250 M
Depth of brace	=	0.500 M
No. of braces	=	4.000

Particular Brace (counted from top	4.000	3	2	1	0	
Design shear	190.842	187.202	183.562	179.922	0.000	0.000
Shear per column	44.530	43.680	42.831	41.982	0.000	0.000
Moment (Kn-m)	68.799	67.486	66.174	64.862	0.000	0.000
Design Moment in brace	134.361	131.774	129.187	127.893	0.000	0.000
Moment of Resitance	160.757	141.181	141.181	141.181	#DIV/0!	
Lever Arm jd	393.240	393.240	393.240	393.240	411.320	411.320
Steel (mm^2) top	1485.556	1456.949	1428.341	1414.038	0.000	0.000
Steel (mm^2) bottom	1485.556	1456.949	1428.341	1414.038	0.000	0.000
Mini. Tension steel required	220.181	220.181	220.181	220.181	220.181	0.000
Provide at top & bottom						
PART-I Dia	20.000	20.000	20.000	20.000	20.000	20.000
Nos.	4.000	4.000	4.000	4.000	0.000	0.000
PART-II Dia	16.000	16.000	16.000	16.000	16.000	12.000
Nos.	2.000	1.000	1.000	1.000	0.000	0.000
TOTA STEEL	1658.761	1457.699	1457.699	1457.699	0.000	0.000
Brace Length	2.825	2.825	2.825	2.825	2.825	2.825
Shear in brace (KN)	105.091	103.067	101.044	100.032	0.000	0.000
Shear stress	0.924	0.906	0.888	0.879	0.000	0.000
% steel	1.327	1.166	1.166	1.166	0.000	0.000
Permissible shear stress	0.400	0.400	0.400	0.400	#N/A	#N/A
Vu	59591.214	57567.470	55543.727	54531.855	#N/A	#N/A
Asv/Sv	0.659	0.636	0.614	0.603	#N/A	#N/A
Provide stirrups dia	8.000	8.000	8.000	8.000	8.000	8.000
Spacing required	150.000	150.000	160.000	160.000	#N/A	#N/A
Provided spacing c/c	150.000	150.000	160.000	160.000	#N/A	#N/A
Spacing as per torsion req	160.000	160.000	160.000	160.000	160.000	160.000

DESIGN OF FOUNDATION

Superimposed Load		=	4493.157 KN		
SELFWEIGHT of Beam		=	179.719 KN		
Net selfweight raft (Assumed)	10.711%	=	481.263 KN		Actual=
Total		=	5154.139 KN		10.511%
Assumed selweight is > Actual Selfweight		=	Hence O.K.		2.447
Soil Bearing Capacity		=	9.500 T/m^2		0.378
Shear at Base for design		=	190.842 KN		
Moment at base for design Seismic F prevail		=	4144.516 KN-m		
Column Circle Dia		=	5.650 m		
Required area of raft		=	54.254 m^2		
Required dia of raft		=	8.311 m		
Provide Outer Dia of Raft		=	8.950 m	Ro	4.475
Inner dia of Raft		=	1.100 m	Ri	0.550
Provided Area of Raft		=	61.962 m^2	Hence O.K.	
Moment of Inertia of raft		=	314.900 m^4		
Section modulus of foundation area		=	70.369 m^3		

Check for wind loads

Moment of wind forces at the base of foundation		=	2388.136 KN m		
Pressure on soil due to wind force ROUND(2388.136/70.369,3)		=	33.937 KN/m^2		
Direct pressure due to superimposed load 5154.139/61.962		=	83.182 KN/m^2	<95	Hence O.K.
Pressure due to direct load (Tank empty and no live load)		=	47.958 KN/m^2		
Pressure due to bending		=	33.937 KN/m^2		
Max. Pressure 117.119		=	117.1 KN/m^2		
Min. Pressure when wind load considerd 14.021		=	14.021 KN/m^2		
max allowable pressure	=1.25*SBC	=	118.750	Hence O.K.	
Udl below foundation		=	75.415		
Over turning moment(Tank empty cond.)		=	4144.516 KN-m		
Restoring moment at edge		=	13297.877 KN-m		
Factor of safety against over turning 13297.877/4144.516		= >	3.209 2.500	Hence Safe	

Check for Seismic loads

Moment of Seismic forces on the base of foundation		=	4144.516 KN-m		
Pressure on soil due to seismic load 4144.516/70.369			58.897 KN/m^2		
Maximum pressure under foundation when tank is full and seismic loads are considerd 83.182+58.897		=	142.079		
Max. Allowable 1.50xS.B.C.		=	142.500 KN/m^2	Hence O.K.	

DESIGN OF RING BEAM BELOW COLUMNS

MIX		=	M 30.000	
Total load		=	4493.157 KN	
Load per Running length w		=	253.136 KN/m	
	Width of Beam	=	450.000 mm	
	Depth of beam	=	900.000 mm	
No. of Columns		=	6.000	
Diameter C/C		=	5.650 m	
C/C Span of beam =PI()*DCC/NC		=	2.958 m	
Clear span of the beam		=	2.508 m	
Effective span of the beam		=	2.958 m	
Span / depth ratio 3.06*1000/700		= >	3.300 2.500	
Hence beam shall be designed as	Ordinery Beam			
Self weight =WFB*PI()*DCC*DFB*25		=	179.719 KN	
Grand Total of weights 4493.157+179.719		=	4672.876 KN	
Load per Running Length w 4672.876/PI()/5.65		=	253.136 KN/m	

For Curved beam					
Angle subtended by one part of beam on the centres of columns					
	2θ	=	36.000 degrees		
Moment Coefficients	C1	=	-0.089		
	C2	=	0.045		
	C3	=	0.009		
Point of Contraflexure is at z		=	12.750 degrees		
Radius R=	9.9/2	=	4.950 M		
Support Moment C1*w*R^2*2θ		=	-188.282 KN-m	As per staad	
253.136*-0.089*(DCC/2)^2*360/NC*PI()/180					
Mid span Moment C2*w*R^2*2θ		=	95.199 KN-m	As per staad	
Twisting Moment C3*w*R^2*2θ		=	19.040 KN-m	As per staad	
Shear Force at Supprot w*R*θ		=	374.430 KN	As per staad	
253.136*DCC/2*PI()/NC					
Shear force at Pt. of contraflexure w*R*(θ − ϕ) =			215.297 KN	As per staad	
Additional Moment due to torsion		=	36.089 KN-m		
=MAX(G1234,J1234)*(1+DRBC/WRBC)/1.7					
Effective support moment		=	188.282 KN-m		
Effective mid span moment		=	95.199 KN-m		
Shear Force at support		=	435.358 KN		
ROUND(374.43+19.04*1.6/(0.45/0.9),3)					
Shear Force at pt. of cotraflexure		=	276.225 KN		
Analysis of beam					
Required effective depth of beam		=	565.953 mm		
SQRT(188.282*1000000 /500 /1.30628)					
Provided over all depth of beam as DFB=900mm is OK		Effective depth of beam =		842.000 OK	
Lever arm z for the beam		=	761.170 mm		
Area of steel required at support			1075.472 mm²		
188.282*1000000/230/761.17/0.904					
Mini. Tension steel required		=	794.494 mm²		
Dia. of bars provided		=	16.000 mm		
Number of bars required					
at bottom of beam		=	6.000 Nos		
		Say	6.000 Nos		
Curtail 50% reinf. at a distance of 0.5D	=	450mm	from the face of support on either side		
Area of steel required at mid span	=	543.781 mm2	MIN Steel=	794.494 mm²	
Dia. of bars provided	=	16.000 mm			
Number of bars required					
at top of beam throughout the length	=	4.000 Say	4.000	Nos.	
Placed within a zone of depth equal to		180.000 mm to the tension face of the beam			

Transverse reinforcement

Shear Reinforcement						
Effective depth	=	842.000 mm				
Shear stress at support	=	1.149 say=	1.149	N/mm²		
435.358/(WFB*(DFB*1000-50-16/2))						
% of steel provided	=	0.298		0.250	0.500	0.245
Permissible shear stress	=	0.245 N/mm²				

As permissiblr shear stress is less than the shear stress at support, shear reinforcement shall be reqd.

Vc	=	99353.891 N		
Vs	=	336004.109 N		
435.358*1000-99353.8905273114				
Asv/Sv	=	1.735		
336004.109472689/(230*842)				
Dia of bar	=	10.000 mm		
Spacing of 2 legged stirrups	=	90.000 mm		
provided spacing	=	90.000 mm c/c	**0.630 m FROM BOTH EDGE**	
			7.000 NOS	
Vs at point of contraflexure	=	176871.109 N		
Asv/Sv	=	0.913		
Providing dia of bar	=	10.000 mm		
For 2 legged stirrups Sv =	=	180.000 mm		
Provided spacing	=	180.000 mm c/c	1.248 m CENTRAL PORTION	
Side face reinforcement	=	405.000 mm²	7.000 NOS	178.286

0.10*Dfb*Wfb*10000

Provided dia. of bars	=	10.000	mm
Number of bars required	=	5.16	no.

Provide 10 mm dia. bars 3 nos. on each face of beam

DESIGN OF FOUNDATION SLAB MIx= M 30.000

Superimposed Load Wf	=	4672.876	KN
From centre of raft, c	=	2.825	m
Inner edge of raft, b	=	0.550	
Outer edge of raft, a	=	4.475	m
Outer edge of beam	=	3.050	m
Inner edge of baem	=	2.600	m
Direct stress	=	75.415	KN/m^2
Concentrated load on beam W	=	4672.876	KN/m
Bending stress	=	33.937	KN/m^2
Stress for design (udl), w	=	75.415	KN/m^3

75.415

Mt==-1/16*w*rx^2+w*b^2/4*(LN(rx/a)+3/4*(-1/3+a^2/b^2+a^2/rx^2)-(a^2+rx^2)/rx^2*(b^2/(a^2-b^2)*LN(a/b)))

Mr=-3/16*w*rx^2+w*b^2/4*(LN(rx/a)+3/4*(1+a^2/b^2-a^2/rx^2)+(a^2-rx^2)/rx^2*(b^2/(a^2-b^2)*LN(a/b)))

Case A : Design of slab as simply supported at edges and carrying udl

Case	Distance r	w (kn/m2)	a (m)	b(m)	c(m)	Circu. Mt (KNM)	Radial Mr (KNM)
	0.550	75.415	4.475	0.550	2.825	539.208	0.000
r < c	0.960	75.415	4.475	0.550	2.825	357.398	176.491
	1.370	75.415	4.475	0.550	2.825	309.644	210.291
	1.780	75.415	4.475	0.550	2.825	287.245	211.327
	2.190	75.415	4.475	0.550	2.825	271.973	198.275
	2.600	75.415	4.475	0.550	2.825	258.729	176.450
	3.050	75.415	4.475	0.550	2.825	244.340	144.723
r > c	3.335	75.415	4.475	0.550	2.825	234.831	120.943
	3.620	75.415	4.475	0.550	2.825	224.840	94.497
	3.905	75.415	4.475	0.550	2.825	214.284	65.483
	4.190	75.415	4.475	0.550	2.825	203.105	33.969
	4.475	75.415	4.475	0.550	2.825	191.264	0.000

Case B : Design of slab as simply supported at edges and carrying udl along the circumference of a concentric circle

Case	Distance r	W(KN/m)	a (m)	b (m)	c (m)	Circu. Mt (KNM)	Radial Mr (KNM)
	0.550	4672.876	4.475	0.550	2.825	574.447	0.000
r < c	0.960	4672.876	4.475	0.550	2.825	381.500	192.947
	1.370	4672.876	4.475	0.550	2.825	333.515	240.932
	1.780	4672.876	4.475	0.550	2.825	314.646	259.801
	2.190	4672.876	4.475	0.550	2.825	305.339	269.108
	2.600	4672.876	4.475	0.550	2.825	300.076	274.370
	3.050	4672.876	4.475	0.550	2.825	294.487	222.967
r > c	3.335	4672.876	4.475	0.550	2.825	285.838	165.179
	3.620	4672.876	4.475	0.550	2.825	274.343	115.689
	3.905	4672.876	4.475	0.550	2.825	261.155	72.516
	4.190	4672.876	4.475	0.550	2.825	246.999	34.283
	4.475	4672.876	4.475	0.550	2.825	232.341	0.000

Mt=Wf/4/PI()*(a^2/(a^2-b^2))*(1+b^2/rx^2)*((LN(a/cr)+0.5-cr^2/2/a^2))

M=Wf/4/PI()*(a^2/(a^2-b^2))*(1-b^2/rx^2)*((LN(a/cr)+0.5-cr^2/2/a^2))

Mt=Wf/4/PI()*(LN(cr/rx)+0.5+a^2/(a^2-b^2)*(1+b^2/rx^2)*(LN(a/cr)+0.5-cr^2/2/a^2)-

Mr=Wf/4/PI()*(LN(cr/rx)-0.5+a^2/(a^2-b^2)*(1-b^2/rx^2)*(LN(a/cr)+0.5-cr^2/2/a^2)+

Design of circular raft with hole

Distance	Circu Mt	Radial Mr	d req, mm	D provided	Steel for Mt		Steel for Mr	
0.550	35.236	0.000	164.244	300.000	694.593		0.000	OK
0.960	24.102	16.456	135.833	325.000	430.919		294.216	OK
1.370	23.871	30.841	153.156	350.000	390.500		501.255	OK
1.780	27.401	48.474	192.635	375.000	413.125		730.839	OK
2.190	33.366	70.833	232.862	400.000	466.502		990.328	OK
2.600	41.348	97.921	273.791	400.000	578.090		1369.054	OK
3.050	50.147	78.244	244.741	400.000	701.112		1100.340	OK
3.335	51.007	44.236	197.605	400.000	713.144		622.095	OK
3.620	49.503	21.192	194.669	375.000	746.352		321.520	OK
3.905	46.871	7.032	189.423	350.000	766.760		115.829	OK
4.190	43.894	0.314	183.309	325.000	784.790		5.654	OK
4.475	41.076	0.000	177.328	300.000	809.668		0.000	OK

Steel Required

	Distance	depth	Mini. Steel	Steel for Mt	Provided dia ϕ	Spacing	Steel for Mr	Provided dia ϕ	Spacing
r < c	0.550	300.000	360.000	694.593	12.000	160.000	0.000	16.000	550.000
	0.960	325.000	390.000	430.919	12.000	260.000	294.216	16.000	510.000
	1.370	350.000	420.000	390.500	12.000	260.000	501.255	16.000	400.000
	1.780	375.000	450.000	413.125	12.000	250.000	730.839	16.000	270.000
	2.190	400.000	480.000	466.502	12.000	230.000	990.328	16.000	200.000
	2.600	400.000	480.000	578.090	12.000	190.000	1369.054	16.000	120.000
	3.050	400.000	480.000	701.112	12.000	160.000	1100.340	16.000	180.000
	3.335	400.000	480.000	701.112	12.000	160.000	1100.340	16.000	180.000
r > c	3.620	375.000	450.000	713.144	12.000	150.000	622.095	16.000	320.000
	3.905	350.000	420.000	746.352	12.000	150.000	321.520	16.000	470.000
	4.190	325.000	390.000	766.760	12.000	140.000	115.829	16.000	510.000
	4.475	325.000	390.000	766.760	12.000	140.000	115.829	16.000	510.000

Provide reinforcement on outside the ring beam part

At Top

Circumferential reinf. as	tor @	0.000 mm c/c	
Radial reinf. as	tor @	0.000 mm c/c	

At Bottom

Circumferential reinf.	12.000 mm dia @	150.000 mm c/c	10.000 NOS
Radial reinforcement	16.000 mm dia @	140.000 mm c/c	Measured at 2.6m from cent

Provide reinforcement at inside the ring beam part

At Bottom

Circumferential reinf.	12.000 mm ϕ@	220.000 mm c/c	10.000 NOS
Radial reinforcement as	16.000 mm dia @	120.000 mm c/c	Measured at 2.6m from cent

The reinforcement will be provided as per the calculations above in the drawings.

Check for Punching Shear

Outer side

Distance of critical section from beam edge =(Ds*1000-50-16/2)/2	=	171.000 mm	Ric=	2.429
			Roc=	3.221
Shear force/m run of section	=	109.438 KN		
Effec.depth of found. at critical section	=	300.960 mm		
Shear stress at critical section	=	0.364 N/mm^2		
Permissible punching shear	=	0.800 N/mm^2		

Inner side

Distance of critical section from beam edge	=	171.00 mm
Shear force/m run of section	=	214.588 KN
Effective depth of foundation at critical section	=	313.472 mm
Sheasr stress at critical section	=	0.685 N/mm^2
Permissible punching shear	=	0.800 N/mm^2

Check for Nominal shear

Effective depth of fouundation at column face	=	342.000 mm
Radius of foundation at critical section	=	3.392 m
Perimeter at critical section	=	21.313 m
Factored load from soil reacton outside critical sect.	=	2018.582 KN
Shear force per meter perimeter at critical section	=	94.713 KN/m
Over all depth of raft at critical section	=	400.000 mm
Shear stress at crtitical section	=	0.240 N/mm^2

Dia of bars provided	=	16.000 mm
Spacing of radial bars available at critical section	=	156.554 mm
Available area of reinf. at critical section	=	1284.299 mm^2
% of steel at critical section	=	0.321 %
Permissible shear stress as per % of steel	=	0.252 N/mm^2
Provided depth of raft is OK		

DESIGN OF STAIRCASE

Mix		=	M 30.000
Width of steps	Wst=		900.000 mm
Tread	T=		260.000 mm
Rise	Rst=		160.000 mm
(R^2+T^2)^0.5		=	305.287 mm
Column circle dia		=	5.650 m
Column Dia		=	450.000 mm
Dia. of Center line of stairs =(DCC+DC/2)		=	5.875 mm
No. of Columns		=	6.000
C/C distence between columns		=	2.825 m
Horizontal length of flight		=	3050.000 mm
Clear Distence between Columns 2.825*1000-0.45*1000		=	2375.000 mm
No. of Tread in one Flight ROUNDUP(2375/260,0)		=	10.000 ACTUAL TREAD width 237.500 mm
No. of Rise 10+1		=	11.000
Vertical distence covered per flight 11*160/1000		=	1.760 m
Length of slab SQRT(3.18^2+1.86^2)		=	2.956 m
Let the thickness of waste slab	Tst=		135.000 mm
Load of one step =(T*Rst/2)*Wst*25/1000000		=	468.000 N
Load of steps/horz.metr ROUND(468*1000/260,3)		=	1800.000 N
Load of slab/horz mtr =(Tst*(Rst^2+T^2)^0.5*Wst*25/1000000)*1000/T		=	3566.571 N
Live Load =3*Wst		=	2700.000 N/m
TOTAL		=	8.067 KN/m
Bending moment ROUND(8.067*2.956^2/12,3)		=	5.874 KN-m
Effective depth required SQRT(5.874*1000000/(Wst*1.11))		=	76.680 mm
Provided depth		=	135.000 mm
Effective depth available 135-20-8/2		=	111.000 mm
Steel required 5.874*1000000/(0.9*111*230)		=	255.647 mm^2
Minimum area of steel required		=	165.904 mm^2
Provided area of steel			
Main bars dia.		=	8.000 mm
			6.000 Say 6.000 Nos
Dist. Bars dia.		=	8.000
Required spacing		=	300.000 mm c/c
Provided spacing		=	270.000 mm c/c

Design of Landing

Superimposed Load	=	23.846 KN
Overhang	=	900.000 mm
Bending moment(add 10%selfweight+ll)	=	12.103 KN-m
Minimum eff. depth required	=	110.068 mm
Provide depth	=	150.000 mm
Effective depth available	=	120.000 mm
Steel required		485.076 mm^2
Main bars dia	=	10.000 mm

no.	=	7.000 nos.	say	7.000	
Height to be covered by steps	=	10.700 m		17.150	
No. of complete flight		6.000			
Load Calculations of Stairs					
Weight of each flight	=	10.076 KN			
Weight of steps per flight	=	5.085 KN			
Weight of each landing	=	3.038 KN			
Total Load		20.655 KN			
Add 5% for railing and other loads	=	21.688 KN			
Total live load on stairs	=	9.949 KN			
Total load of stairs includg. live laod	=	284.732 KN			
Load shared by each column	=	31.637 KN (for critical load condition)			
Level of first flight	=	4.500 m			
Height gained in one flight	=	1.760 m			
Total no. of flights provided	=	6.000 Nos			
Level of last flight	=	15.060 m	Lcu=	3.590	
Height left below the lower edge of cone wall	=	2.637 m			
Level of balcony	=	19.100 m	actual	19.100	
Height of ladder portion	=	4.040 m			
TOTAL LOAD OF STAIRS	=	27.642			

Design of RCC Ladder

Grade of concrete	=	M 30.000		
Clear width of ladder	=	0.600 m		
Center to center width of ladder	=	0.725 m		8.300
Section of ladder beam	b =	0.125 m		5.650
	D=	0.250 m		0.150
Section of steps	b =	0.250 m		1.050
	D=	0.080 m		
C/C distance of steps in vertical direction	=	0.200 m		
Vertical height of ladder	=	4.040 m		
Horizontal distance covered by ladder	=	2.150 m		
Length of ladder	=	4.576 m		
Self weight of ladder beam	=	7.151 KN		
Number of steps in the ladder	=	21.000 Nos		
Load of steps	=	7.613 KN		
Live load	=	4.394 KN		
Total load on beam	=	19.157 KN		
Total load on beam per m	=	4.742 KN/m		

Design of beam

Bending moment	=	6.449 KN -m
Assuing dia of bar to be used	=	16.000 mm
Effcetive depth of beam required	=	215.598 mm
Total depth of beam required	=	243.598 mm
	Assumed depth of beam is correct	
Area of steel required	=	139.724 mm^2
Numcer of bars required	=	1.000 Nos
Provided number of bars	=	2.000 Nos.

Provide nominal stirrups of 8 mm dia @ 250 c/c

Design of Steps

Design load on step	=	2.394 KN/m
Effective span of step	=	0.680 m
Bending moment	=	0.138 KN m
Effective depth of step required	=	22.332 mm

Assumed depth of step is correct

Dia of bars used	=	8.000 mm
Effective depth of step available	=	41.000 mm
Area of steel required	=	16.234 mm^2
Number of bars required	=	1.000 Nos
Provided number of bars	=	2.000 Nos

SEISMIC CALCULATION

Volume of water		192.87 cum	8.45		
Dia of Cyl wall	IDT=	8.3 m		TWAe=	
Equivalent height of HTWe=		3.56 m	Equ Thickness of CYL wall		0.235
	h/D=	0.46506024			

CSCWorld
######
Location of site : TRAMBAY

i)	Total weight of top dome	=	227.54 KN
g)	Total weight of ring beam	=	33.575771 KN
f)	Total weight of vertical wall	=	306.60257 KN
f)	Total weight of balcony bean	=	108.23768 KN
f)	Total weight of conical wall	=	168.30706 KN
l)	Total weight of top dome	=	117.49557 KN
i)	Total weight of ring beam	=	199.68746 KN
		TOTAL	1183.4461 KN
		TOTAL WITH	983.75866 KN

Self weight of tank,(W1)= (a) + (b) + (c) + (d) + (e)
1183.44614
Water load,(W2)= 1675.87
(f) Self weight of staging beams= 342.576698 KN
(g) Self weight of columns= 264.84375 KN
STAIR + PIPING 353.497649
Total weight of staging,(W3)= (f) + 960.920297

CASE I : TANK EMPTY CONDITIONTotal Mass,(M)= Ms
Where,
Ms= Structural Mass
= W1 + ((1/3) x W3)

1503.753
Equivalent spring of the syst = 19653.084 KN/m
16377570.266*NC/(NB+1)/1000

The damping in the concrete structure is assumed to be 5%
Earthquake force in X direction :
Delta when tank empty = 0.077 m
Fundamental period when ta = 0.5566609 sec
2*PI()*SQRT(0.077/9.81)
Importance factor,(I)= 1.50
Refer fig. 2 of IS 1893 with 5 SaX/g =
Avg. acc. coeff when tank er = 2.5
alpha-hX= Z/2 x I/R x (SaX/g)
Horz. seismic coeff when tank is
Empty = 0.0625
0.1*1.5*1*2.5/2/3
Lateral force = alpha-hX x W
93.98455669

Earthquake force in Y direction :

Same as in x direction

CASE II : TANK FULL CONDITIONTotal mass,(M)= Mi + Ms
where
Mi= Impulsive Mass

Where Mi is Calculated from
Mi/m =(tanh 0.866 D/d)/(0.866 D/h)
(Section 4.2.2.2)
m= Mass of Water m 1675.87 KN
Ms = Structural Mass Mi 1007.3617 KN
= W1 + ((1/3) x W3)
1503.753 KN

Mc= Convective Mass

Where Mc is Calculated from

Mc/m =0.23(tanh(3.68h/D))/(h/D) 869.11715 KN
(Section 4.2.2.2)

a) Impulsive mode
Time period Impulsive = natural period of the structure , in seconds 0.7170757
SaX/g Impulsive= 1.67
Avg. acc. coeff when tank fu = 2.35

b) Convective Mode h/D= 0.46506024 Cc= 3.555
Fundamental period when tank full = 3.2699729 sec
Cc*SQRT(8.3/9.81)
SaX/g convective= 1.67
Avg. acc. coeff when tank fu = 0.51
h= Depth of water
D= Inner diameter of tank
Importance factor,(I)= 1.50

alpha-hX Impulsive= Z/2 x I/R x (SaX/g)i 0.059

SaX/g Convective=
alpha-hX Convective= Z/2 x I/R x (SaX/g)c 0.013

Lateral force Impulsive Vi= alpha-hX (I) x (Mi+M 147.65 KN

Lateral force Convective Vc= alpha-hX (c) x Mc 11.12 KN

Total Lateral force V= √(Vi²+Vc²) 148.07 KN

Earthquake force in Y direction : .

Same as in x direction 148.07 KN

Value of αh

Average acceleration coeff. (Sa/g)i	0.717
Average acceleration coeff. (Sa/g)c	3.270
αhi=Z/2*I/R*(Sa/g)i	0.059
αhc=Z/2*I/R*(Sa/g)c	0.013

Total pressure on wall is given by
P=Static(water)Pressure(Pw)+Hydrodynamic Pressure(Ph)
(If Ph>1/3 *Pw, then P=Pw+1/3*Ph. Otherwise P=Pw.)
Static Pressure on wall is given by
Pw=w*y
Maximum pressure on wall is given by
P=sqrt((Piw+Pww)^2+Pcw^2+Pv^2)
Impulsive hydrodynamic pressure on wall
Piw=QiwX(Ah)iXwxgxhxcos φ
Qiw=(1-(y/h)^2)tanh(0.866D/h)0.886
Convective hydrodynamic Pressure on wall
Convective hydrodynamic Pressure on wall is given by
Pcw=QcwX(Ah)cXwxgxhx(1-1/3cos^{2}φ)cos φ
Qcw=0.5625 cosh(3.674 y/D)/cosh(3.674h/D)
Pressure on wall due to Inertia
Pressure due to wall inertia is given by
Pww=αhi*t*ρ*g
Pressure on wall due to Vertical Excitation
Pressure due to Vertical excitation is given by
Pv=αv*w*g*h(1-y/h)
αv=(2/3)*Z/2*1/R*(Sa/g)

where

α hi=horizontal seismic coeeficient for impulsive mode

α hc=horizontal seismic coeeficient for convective mode

w= unit weight of water=1000 kg/m^3
h= height of wall=3.86 m
$\varphi = 0$=circumferential angle
y=height from top
Z=Zone factor=0.1
I=importance factor=1.50
R=seismic Reduction Factor=3.00
Sa/g=Average acceleration coeff.

Internal diameter of tank	=	8.300 m
Water depth for design purpose	=	3.860 m
HTWe+0.3		(Including Free Board)
Net Height of wall	=	3.860 m

Dist(h-y) from top (M)	y/h	water pressure(cal) (KN/m2)	Hydro dynamic pressure impulsive Piw=KN/m2	Hydro dynamic pressure convective Pcw=KN/m2	Pressure due to wall Inertia ww=KN/m	Pressure due to vertical excitation Pv=KN/m2	EQUIV Thicknes (mm)	maximum Hydrodynamic pressure P	Provide Thickness (mm)	H^2/Dt
0.000	1.000	0.000	0.000	1.538	0.221	0.000	235	1.554	150.000	11.968
0.386	0.900	3.860	0.364	1.313	0.221	0.151	235	1.445	150.000	11.968
0.772	0.800	7.720	0.690	1.127	0.221	0.303	235	1.460	150.000	11.968
1.158	0.700	11.580	0.977	0.973	0.221	0.454	235	1.609	150.000	11.968
1.544	0.600	15.440	1.226	0.848	0.221	0.605	235	1.783	150.000	11.968
1.930	0.500	19.300	1.437	0.748	0.221	0.757	235	1.970	150.000	11.968
2.316	0.400	23.160	1.610	0.670	0.221	0.908	235	2.150	150.000	11.968
2.702	0.300	27.020	1.744	0.612	0.221	1.059	235	2.314	150.000	11.968
3.088	0.200	30.880	1.840	0.571	0.221	1.210	235	2.457	150.000	11.968
3.474	0.100	34.740	1.897	0.547	0.221	1.362	235	2.576	150.000	11.968
3.860	0.000	38.600	1.916	0.539	0.221	1.513	235	2.673	150.000	11.968
						Average		2.001	KN/sqm	

sloshing wave height(dmax)
=(Ah)c*R*D/2
0.159
Height of sloshing wave is less than free board of = 0.300

Dist from top (M)	Distance from bottom (M)	Hoop static force (IS TAB) (KN)	Hoop static force(cal) (KN)	Hoop static force(max) (KN)	Tension due to dynamic forces(KN)	Total tension (KN)	Steel Required (mm^2)	Min. Thickness required (mm)	Provide Thickness (mm)
0.000	3.860	0.720	0.000	0.720	6.560	7.280	55.996	4.984	150.000
0.386	3.474	14.336	16.310	16.310	6.110	22.420	172.462	15.349	150.000
0.772	3.088	29.872	32.620	32.620	6.250	38.870	299.000	26.611	150.000
1.158	2.702	46.148	48.930	48.930	6.800	55.730	428.692	38.154	150.000
1.544	2.316	63.419	65.230	65.230	7.530	72.760	559.692	49.813	150.000
1.930	1.930	80.218	81.540	81.540	8.320	89.860	691.231	61.520	150.000
2.316	1.544	92.685	97.850	97.850	9.080	106.930	822.538	73.206	150.000
2.702	1.158	97.597	114.160	114.160	9.780	123.940	953.385	84.851	150.000
3.088	0.772	72.725	130.470	130.470	10.380	140.850	1083.462	96.428	150.000
3.474	0.386	31.020	146.780	146.780	10.890	157.670	1212.846	107.943	150.000
3.860	0.000	0.000	163.090	163.090	11.290	174.380	1341.385	119.383	150.000

Average Hydrodynamic pres = 2.001 KN/sqm
Base Shear at column top = 64.108038 KN

2.001*PI()/4*8.3^2*(3.56+0.3)/2
Base Shear at per column = 10.685 KN
h/D= 0.4650602
hi= 0.375
Base Moment at per colum = 15.466064 KN-m

CONTINUITY ANALYSIS

(a) TOP DOME NAHRI

ψd=	2pR1sinϕ Et1	=	318166.6667 /E	221333.3
δd=	pR1^2*sin(ϕ)/Et1	=	-234143.6277 /E	180
M=	ER1t1/(4α1^3)*(K1+1/K1)	=	0.000742705 E	20
α1=	3(R1/t1)^0.25	=	8.769436742	
K1=	1-Cot(ϕ)/2/α	=	0.928435142	
H=	Et1/(αR1K1sin^2(ϕ))	=	0.007124361 E	
M/un=Et1/2α1^2K1sinϕ		=	0.001685746	
H/un=M/un				
E=			27386127875	

(B)Ring BEAM

H=	Ebd/R^2	=	0.002735865 E
M=	Ebd^3/12R^2	=	0.0000091 E

(C) C WALL

α=	(12/D^2/T^2)^.25	=	1.65316451 E
Z=	ET^3/12	=	0.00028125
M=	2αZ	=	0.000929916 E
H=	2α^2*Z	=	0.001537323
M=/un	2α^2Z	=	0.001537323 E
H=/un	2α^3*Z	=	0.005082956
ψω=			1166240.833
δω=			3615346.583

	mem def	net def		Clockwise moments M		Inward Thrust H	
1.DOME 1.out mov	-234144 /E mt.	234143.628	/E+y1	-0.001685746 xEy1+	-394.707	0.007124 xEy1+	1668.124
2.clw.rot	318166.7 /E rad.	-318166.67	/E+z1	0.000742705 xEz1+	-236.304	-0.00169 xEz1+	536.3483
2.BEAM			y1			0.002736 xEy1	
			z1	9.11962E-06 xEz1			
3.WALL			y1	0.001537323 xEy1		0.005083 xEy1+	
	1166241 /E rad.	-1166240.8	/E+z1	0.000929916 xEz1+	-1084.51	0.001537 xEz1+	-1792.889
4.REACTION DUE TO THRUST FROM DOME							
TOTAL				-0.000148423 xEy1+ 0.001681741 xEz1+ -1715.517058		0.014943 xEy1+ -0.00015 xEz1+ -16400.2	-16811.75

y1= 1106603.663

z1= 1117924.662 /E

8.00

MEMB	AREA OF MEMB	CLOCKWISE ROT(NET)	DISPLACE(NET)	THRUST BORNE	MOMENT	HOOP TENSION
TOP DOME	0.15	799758	1342747.49	8218.028916	-1669.547489	-39358.7
BEAM	**0.05**	**1117924.7**	**1106603.66**	**3033.012177**	**10.19504269**	**-12966.1**
WALL	0.15	-48316.171	1117924.66	5560.709094	1659.352446	-39358.7

A)=	BOTTO RING BEAM				
	M=(bd^2E/2Ro^2		0.01879552		
Correspond H=	H=bd^2E/3Ro^2		0.028193281		
Thrust stiffness	H=bdE/Ro^2		0.056386561		
Corresp moment	=bd^2E/2Ro^2		0.028193261		
B)=	BOTTOM DOME	Rabd=	4.40	SIN(Xb)=	0.580909
ψd=	2pR2sinφ Et2+0-wR2^2Sin F2/Et2	=	-430560 /E		
δd=	pR1^2*sin(φ)/Et1	=	-1415164.872 /E		
M=	ER1t1/(4α1^3)*(K1+1/K1)	=	0.001683446 E		
α2=	3(R2/t2)^0.25	=	6.172934291		
K2=	1-Cot(Φ)/2/α	=	0.889416514		
H=	Et2/(αR2K1sin^2(φ))	=	0.023710414 E		
M/un=Et1/2α1^2K1sinφ		=	0.004993336		
H/un=M/un					
a)CONICAL DOME					
		W	106960.2009		
HOOP TENSION AT BOT=			186632	HT AT BOT OF CONE WALL f216	
MERID THRUST AT BOT=			142548	MERID THR IN CON f208	
ANGLE			0.722		
		DT	6.50		
		OB	6.30		
		SL	4.766322		
		=	/E		
δd=AT TOP		=	2939454 /E		
ψd=AT TOP	-TAN(F)(2T2+T1)/Et		-2271152.78		
M=	ER1t1/(4α1^3)*(K1+1/K1)	=	E		
z1=	2*ΔL^.5	=	19.38464507		
Δ1=	(12/(TAN(F))^2)/t^2)^.25	=	4.434941152		
		K1	1305.2		
		K2	93.62		
		K3	93.62		
		K4	13.888		
M=	Et1K4L/(K1K4-K2K3)TAN²(φ))	=	0.001631135 E		
H=	Et1K2L/(K1K4-K2K3)SIN(F)TAN(φ)	=	0.0034587		
H/un=	Et1K1L/(K1K4-K2K3)L*SIN2(F))	=	0.013451442		
M/UN=		=	0.0034587		

==

	mem def	net def	Clockwise moments M		Inward Thrust H	
1.DOME2 out mov	1415165 /E mt.	-1415164.9 /E+y3	0.004993336 xEy3+	-7066.39	0.02371 xEy3+	-33554.14
clw rot	-430560 /E rad.	430560 /E+z3	0.001683446 xEz3+	810.9363	0.004993 xEz3+	2149.931
2.BEAMB2		y3	-0.028193281 xEy3		0.056387 xEy3	
		z3	0.01879552 xEz3		-0.02819 xEz3	
3.CON DOME	-2939454 /E mt.	2939454 /E+y3	0.0034587 xEy3	10166.69	0.013451 xEy3+	39539.9
	-2271153 /E rad.	2271152.78 /E+z3	0.001631135 xEz3+	4158.786	0.003459 xEz3+	7855.236
4.NET THRUST FROM CONICAL DOME & dome			NET RADIAL THRUST =THc-THd f266			-16277.21
TOTAL			-0.019741244 xEy3+		0.093546 xEy3+	
			0.022510101 xEz3+		-0.01974 xEz3+	
			8070.017055		-286.292	

y3= -89080.2471

z3= -436629.413 /E

==

MEMB	AREA OF MEMB	CLOCKWISE ROT(NET)	DISPLACE(NET)	THRUST BORNE	MOMENT	HOOP TENSION
CONE	0.2	1834523.4	2850373.75	44686.70345	13217.84606	5655.889
BEAM BOT	0.45	-436629.41	-89080.247	7287.066759	-5695.21264	14189.77
DOME	0.2	-6069.4132	-1504245.1	-35696.5806	-7522.633421	6306.566

	4.475						PLACE :	TRAMBAY
	0.55	2.825	3.2375	3.65	4.0625	4.475		
	2.825	0.55	1.308333	2.066667	2.825			
	0.4						g_w=	9.81
	0.3							

Levels

RBC	21.54
Brace 4th	17.95
Brace 3th	14.36
Brace 2th	7.18
Brace 1th	3.59
Brace 0th	
FDN BEAM	0

Loads To be considered in staad

S.NO	Dead loads			
1)	Self Weight excluding RBC of elemenents	=	982.72 kN	199.6875
	DL Per meter length on tank	=	**55.4 kN/m**	
	Self Weight including RBC of elemenents		**1182.41**	
			66.6 kN/m	LOAD 1 LO
S.NO	**Live loads**			
1)	Live Load on tank	=	270 kN	
	LL Per meter length on tank	=	**15.228 kN/m**	
S.NO	**Water loads**			
2)	Weight of Water on tank	=	1875.87 kN	
	water weight Per meter length on tank	=	105.669 kN/m	

1.5.4 Wind Load

Basic wind speed (As per Figure -1 of IS : 875 (Part 3) - 1987) Vb = 47 m/sec
Design Wind Speed is given by Vz = K1xK2xK3xVb
Factor K1 = 1.07
(For important building and structure, as per Table -1 of IS : 875 (Part3)- 1987)
Factor K2 (As per Table -2 of IS : 875 (Part 3) - 1.1
Factor K3 = 1
(Topography factor, as per Clause 5.3.3.1 of IS : 875 (Part 3) - 1978)
Design wind speed = Vb x K1 x K2 x K3 = 47*1.07*1.1*1 = 55.319 m/sec
Design wind pressure = $0.6V_z^2$ = 0.6 * 55.319 = 1836.115 N/Sq.Mt
Net wind pressure on circular shapes = Cf * pz =0.7 * 1836.115 = 1285.281 N/Sq.Mt
Wind load on column = 1285.281 * 450 = =0.5784kn/m.
Wind load on Brace beam =1836.115* 500 =0.9181kn/m.

1.5.5 Earth quake Load (EQ)

TRAMBAY is in Zone II as per the IS 1893 part1 2002
Z=0.1, I=1.50, R=3.00 from table 7 of IS 1893 part1 2002
S.S = 1 for Hard soils, ST = 1 For RCC Structures.
S_a / g and ZIRS$_a$ / g calculated in analysis in which the above data is considered. Seismic analysis is

Weight of container

982.72 KN	
Weight of one brace	8.828125 KN
Weight of Column	14.27412 KN

JOINT WEIGHT CALCULATIONS

Joint weight at level of RBC	163.7871 KN	JOINT WEI
WIND Force excl RBC		
Moment on RBC	39.26233 KN-m	
Wind shear on RBC	10.21751 KN	
Shear Force due to container water	10.685 KN	

INTERACTION RATIO: 0.41 (as per Cl. 39.6, IS456:2000)

SECTION CAPACITY BASED ON REINFORCEMENT PROVIDED (KNS-MET)

WORST LOAD CASE: 110
END JOINT: 12 Puz : 2349.13 Muz : 118.69 Muy : 119.17 IR: 0.65

Checked SR 190KL Staging 18m.
Trambay, Ajmer.
Ravi Saxena
6/8/17

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30/8/17